Pavement Performance Modeling

of

Unique Crosswalk Designs

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

Interlocking Concrete Pavements also known as block pavements are one of the integral parts of the pavement system in Europe. The use of ICP slowly extended to other countries including North America. As the usage increased the need for more scientific research was developed which resulted in the study of ICP design and analysis methods, construction practices and materials specifications.

This thesis presents a research project involving the design, construction, instrumentation, performance modeling and other field tests of eight ICP crosswalks with four different design assemblies. The research projects were constructed at the Centre for Pavement and Transportation Technology (CPATT) Test Track and at the University of Waterloo Ring Road. Each of the test sections is instrumented with structural and environmental sensors of sensors to monitor the pavement performance under heavy truck traffic, typical municipal loadings and to quantify environmental effects.   A database is generated and the measured stress, strain, temperature and moisture measurements are analysed to evaluate the expected long-term performance of the structural components of ICP crosswalk designs.

Acknowledgements

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I gratefully appreciate the financial support of the Interlocking Concrete Pavement Institute (ICPI) for the successful completion of this work. I would like to gratefully acknowledge the guidance and technical support of Robert Burak, P. Eng, former Director of Engineering, ICPI, David Smith, P. Eng, Technical Director, ICPI and Robert Bowers, P.Eng, Director of Engineering, ICPI. In addition, special appreciation is extended to Ross Yantzi, Steve Lange, Jason Bell from Ross Yantzi's Pavestone Plus Limited, Gary Kosar from the University of Waterloo Plant Operations, and Cameron Becker from Steed and Evans for their assistance in construction and instrumentation.

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Dedication

I would like to dedicate this thesis to my parents and brother for supporting me throughout the years of my academic study and professional career.

I would like to thank my loving husband Ravi for his support, encouragement and reinforcement throughout my graduate studies.

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Chapter 1

Introduction and Objective

1.1 Background

The concept of interlocking concrete pavement dates back to the roads of the Roman Empire. They were constructed with tightly-fitted stone paving units set on a compacted aggregate base. The modern version, concrete pavers, is manufactured with close tolerances to help ensure interlock. Concrete pavers were developed in the Netherlands in the late 1940's as a replacement for clay brick streets. A strong, millennia-old tradition of segmental paving in Europe enabled interlocking concrete pavement to spread quickly. It is now established as a conventional means of paving there with some three billion ft^2 (300 million m^2) installed annually. Concrete pavers came to North America in the 1970's. They have been used successfully used in residential, commercial, municipal, port and airport applications (ICPI 2006).

The Interlocking Concrete pavement Institute (ICPI) in partnership with the Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo planned and designed an ambitious research project that would validate and evaluate the structural performance of different crosswalk designs on two different test sections. Monitoring the two different pavement test sections were facilitated by using sensors that are capable of capturing the strain, vertical pressure, moisture content and temperature. Installation of these sensors was also part of this research and involved careful selection and design of sensors and gauges so that data collected would enable the researchers to monitor and structurally evaluate the performance of the different designs. To facilitate with the research, other tests like Falling weight deflectometer, Profiler and distress survey were also used.

1.2 Research Scopes and Objectives

The main objectives of the research project are:

- To define the mechanics of failure for ICP crosswalk designs with various bases and setting beds.
- To quantify the threshold value for type and/or number of axle loads (ESALs) for various crosswalk assemblies.
- To validate current industry crosswalk designs and recommend new designs (or modifications to existing designs) as needed based upon the load/traffic/environment/failure modes from the study.
- To offer the designer/city/municipality/DOT at a higher level of confidence on long term crosswalk performance.

In short the research will offer design professionals guidance on design protocols and performance of ICP crosswalks for various loading conditions.

1.3 Methodology

The methodologies followed in this research project to achieve the objectives are discussed in the following section.

For the purpose of assessing structural performance of different crosswalk designs of ICP, seven crosswalks with different bases and bedding layers were constructed at two locations in Waterloo. The first four sections are located at the Centre for Pavement and Transportation Technology (CPATT) Test Track in the south east corner of the Regional Municipalities of Waterloo Waste Management Facility. The second test site with four test sections is located at North Campus Gate intersection on the University of Waterloo Ring Road. The test sections are instrumented with four sets of sensors at the Test Track and three sets of sensors on the Ring Road, to monitor the pavement performance under heavy truck traffic, typical municipal loadings and to quantify environmental effects. The sensors include vibrating wire strain gauges, earth pressure cells, and temperature and moisture probes. Data was collected at four-hour intervals and includes stresses, strains and temperature. Moisture data was collected on two times a month. A database is being generated for all eight sections and the measured stress, strain, temperature and moisture measurements are analysed to evaluate the expected long-term performance of the structural components of ICP crosswalk designs. Traffic data at Test Track was obtained from the Region of Waterloo waste management automation system. Pavement predictive models are developed based on measured structural and environmental parameters.

Also, routine performance testing including distress (type, severity, density) survey was carried out on a regular basis in accordance with Interlocking Concrete Block Pavement Institute Distress Guide developed by ICPI. The total of eleven types of surface distresses evaluated separately for each test sections. The degree of distress is rated high, medium, or low based on Pavement Condition Index (PCI) numerical indicator. Other surveys like Portable falling weight deflectometer testing at the centre of the crosswalk and at the edge and Profile testing was done on the wheel paths of the crosswalk with SurPRO.

1.4 Thesis Organization

This thesis consists of seven chapters, and the contents of each chapter are explained as follows: Chapter one provides and introduction to the research project. A general overview about the thesis scope and objectives is provided. In addition, the research methodology is explained.

Chapter two provides a literature review that covers the history of development of interlocking concrete pavements, advantages and limitations, components of ICP and their affect in performance, design methodology.

Chapter three explains the site description, pavement structure, and the crosswalk construction of eight test sections at two locations. This chapter also covers the instrumentation at the crosswalk sections.

Chapter four presents the repair and maintenance in the crosswalk sections.

Chapter five presents the performance of the crosswalk sections. This chapter provides the pavement response under traffic and environmental loadings including pavement distress condition survey, deflection results and profile of the sections.

Chapter six presents the life cycle cost analysis of the four crosswalk designs.

Chapter seven summarizes the main conclusions of this research and recommendations including the ranking of the crosswalks according to the performance.

Chapter 2

Literature Review

Unlike other paving techniques, Interlocking Concrete Pavement (ICP) is the form of pavement in which the wearing surface is made from small paving units bedded and jointed in sand rather than continuous paving. Beneath the bedding sand, the substructure is similar to that of a conventional flexible pavement. ICP is comprised concrete blocks and is bedded and jointed in sand. Interlock has been defined as the inability of an individual paver to move independently of its neighbours and has been categorized as having three components: horizontal, rotational and vertical. Interlock is of major importance for the prevention of movement of pavers horizontally when trafficked (Knapton 1979).

2.1 Development of Interlocking Concrete Paving

Segmental paving dominated the world's urban pavement construction from the middle ages. This type of paving was based on the use of wooden or stone setts and brick or cobblestone which were laid by hand. The concept of modern interlocking concrete block pavement began in Europe, specifically in Holland and Germany. The first significant trial of block paving appears to have been at Neuss, Germany in 1936 when rectangular units 240 x 120 x 80 mm were successfully traffic used under heavy traffic. The earliest widespread uses of paving units were in Holland. In the places like footpath, where wear and tear is less, paving units made from locally available material clay were used. After the introduction of rubber tyre, the use of bricks as a paving material extended from Footpath to road pavements. In these brick pavements the traffic lanes were delineated by specially manufactured concrete blocks similar in shape to bricks. After the world war II full-scale production of the concrete blocks begun and were widely used (Shackel 1980). The prime reason for retaining segmental paving in Holland is also because of the regular maintenance to be done. Historically, the rectangular units proved to be capable of accommodating large deformations without damage and hence remain the dominant shape (Kellersmann 1980).

The success of block paving in Holland and Germany was soon repeated in neighbouring countries such as Belgium, Denmark and Austria. The many advantages and versatility of interlocking blocks caught the attention beyond Europe. In the 1960s it made its appearance in Brazil, Argentina and South Africa. And by 1970s interlocking concrete block paving became established in North America, Britain, Japan, Australia and New Zealand. Amongst the non-European countries, Australia is successful in the range and quality of block paving applications.

Outside Europe, the block paving was restricted to be appropriate only in local conditions, which resulted the concrete block pavements being considered for only architectural applications. The scientifically based design information was necessary to use in roads. For this reason, the research was first initiated in South America and South Africa which were followed by other nations.

2.2 Advantages and Limitations of Interlocking Concrete Paving

2.2.1 Advantages

The historical experience worldwide has established that interlocking concrete pavers possess a number of advantages. The advantages are not only limited to the earlier forms of segmental paving, but also for certain applications, over conventional flexible and rigid pavements. The advantages are listed as follows:

- ICP make no use of expensive imported materials such as Bitumen.
- Interlocking blocks may be cheaply mass-produced to high dimensional and strength tolerances.
- ICP are permeable in nature.
- ICP may be opened to traffic immediately after construction (i.e. the delays associated with the curing of conventional rigid pavements are avoided).
- Maintenance costs are low because it is possible to replace readily any areas of local failure in the blocks.
- ICP can be repeatedly lifted and re-laid at minimum cost that ready access is provided to underground services and that trenching and reinstatement of the pavement is both easier and less unsightly than in conventional pavements.
- They are aesthetically acceptable in a wide range of application, and can be used for odd-shaped areas.
- Unlike bituminous surfacing block pavements have a high resistance to fuel and oil spillages.
- Line markings and traffic control markings can be permanently incorporated in the block pavements by the suitable use if colour.

2.2.2 Limitations

There are few limitations of Interlocking Concrete block paving. They are listed as follows:

- Traffic speed on the block pavements should be limited to 70km/hr. Neither the riding quality nor the skid resistance of block pavements is appropriate to high speed traffic operation.
- The other limitation of block pavements is their roughness.
- ICP cannot be laid as rapidly as machine laid rigid pavements.
- Surface maintenance to minimize clogging to ensure long-term performance

2.3 ICP Components

An ICP is a flexible pavement in which the surfacing consists of concrete pavers laid on a thin layer of sand referred to as the laying course or bedding sand (Beauty 1992). The base layer can be constructed using untreated aggregate, asphalt treated base or cement treated base. If either an asphalt base or cement treated base is used a granular subbase layer maybe placed underneath the treated base layer.

2.3.1 Interlocking Concrete Paver (ICP)

Interlocking Concrete Pavement (ICP) is the form of pavement in which the wearing surface is made from small paving units bedded and jointed in sand rather than continuous paving. It provides a hard surface which is good from the aesthetic point of view, comfortable to walk on, extremely durable and easy to maintain. It adds a richness, complexity and human scale to any setting. The concrete paving blocks used typically are 200 mm to 250 mm long and 100 mm to 112 mm wide. The thickness of the blocks used ranges from 60 mm to 100 mm, depending on the traffic intensity. The paving blocks are typically installed on a sand bed 20 mm to 40 mm thick, separated by sand joints of 2 mm to 4 mm.
Accelerated trafficking studies in Australia, South Africa, Japan, France and the USA have shown that pavement performance is influenced by the shape of the pavers in respect of both horizontal and vertical deformations. Essentially, the blocks are assigned to three different categories which are defines as follows:

Category I: This category comprises dentate blocks which key into each other on all four faces, and these units are capable of being laid in herringbone bond.

Category II: Category II comprises of dentated blocks which key into one another on two faces only and these units are capable of being laid in stretcher bond.

Category III: This comprises non-dentated blocks which do not key together geometrically, but rely on their dimensional accuracy and accuracy of laying to develop interlock (Shackel 1986).

There are a number of different laying patterns for interlocking concrete pavement used all based on three basic bonds, Stretcher, Stack and Herringbone. The Herringbone pattern should be adopted for rectangular blocks if the pavement is to carry vehicular traffic, as this prevents the block creeping horizontally, under the action of accelerating and braking vehicles. Most of the blocks lock together on their sides as a gear does. This "gearing" limits relative horizontal movement which allows block to be in non-herringbone patter, even when there is vehicular traffic (Lilley 1994).

2.3.2 Bedding Sand and Jointing Sands

Bedding sand and Joint sand are one of the most important components of ICP. Bedding sands are a critical component of all sand-set segmental concrete paving systems. Especially for vehicular applications, specifiers and contractors need to consider bedding sand selection. While gradation is an important consideration, other characteristics should be assessed in order to ensure long term pavement performance. Bedding sand provides four main functions. It beds the pavers during installation; helps initialize interlock among the pavers; provides a structural component for the system and facilitates drainage of water that infiltrates through the joints (ICPI Tech Spec 17 2007).

The laying course is considered to be an essential element, in a concrete block pavement, which facilitates placing the pavers. The main functions of the laying course are: (Beaty 1992)

- To provide an even surface on which to lay the blocks
- To accommodate accepted tolerances in the finished surface level of the base
- To provide uniform support to the blocks and to avoid stress concentrations which could cause damage to the blocks.
- To fill the lower part of the joint spaces between adjacent blocks in order to develop interlock

The gradation of bedding Sand according to ASTM C 33 and CSA A23.1-FA1 is provided in the table 1 below.

Table 1 Grading Requirements for Bedding Sand

ASTM C 33		CSA A23.1-FA1	
Sieve Size	**Percent Passing**	**Sieve Size**	**Percent Passing**
3/8 in. (9.5 mm)	100	10 mm	100
No. 4(4.75 mm)	95 to 100	5 mm	95 to 100
No. 8 (2.36 mm)	80 to 100	2.5 mm	80 to 100
No. 16 (1.18 mm)	50 to 85	1.25 mm	50 to 90
No. 30 (0.600 mm)	25 to 60	0.630 mm	25 to 65
No. 50 (0.300 mm)	10 to 30	0.315 mm	10 to 35
No. 100 (0.150 mm)	2 to 10	0.160 mm	2 to10
No. 200 (0.075 mm)	1	0.075 mm	1

Jointing sand plays a major role in promoting load transfer between blocks ultimately in spreading the load to larger areas in lower layers. Spacing between pavers showed a significant influence on the structural performance of block surfacing. Pavers laid too close together become overstressed and spall when in contact. However, the structural capability of the surfacing decreases if the joint width exceeds 5 mm (Lilley, 1994). Knapton and O‘Grady (1983) recommended joint widths between 0.5 and 5 mm for better pavement performance. Joint widths ranging from 2 mm and 8 mm are often used, depending upon the shape of pavers, laying pattern, aesthetic considerations, and application areas.

In most of the pavements, the sand used for bedding course is also used in joint filling. A finer jointing sand having a maximum particle size of 1.18 mm and less than 20 % passing the 75 µm sieve has performed well (Shackel 1980).

In the cases where there is channelization of heavy vehicle loads, the application of joint sand in the newly constructed pavement is recommended in order to minimize the ingress of water (Beaty 1992)

The Gradation of Joint Sand according to ASTM C 144 and CSA A179 is provided in the table 2 below.

Table 2 Grading Requirements for Joint Sand

ASTM C 144			CSA A179	
Sieve Size	**Natural Sand Percent Passing**	**Manufactured Sand Percent Passing**	**Sieve Size**	**Percent passing**
No. 4(4.75mm)	100	100	5mm	100
No. 8 (2.36mm)	95 to 100	95 to 100	2.5mm	90 to 100
No. 16 (1.18mm)	70 to 100	70 to 100	1.25mm	85 to 100
No. 30 (0.600mm)	40 to 75	40 to 100	0.630mm	65 to 95
No. 50 (0.300mm)	10 to 35	20 to 40	0.315mm	15 to 80
No. 100 (0.150mm)	2 to 15	10 to 25	0.160mm	0 to 35
No. 200 (0.075mm)	0 to 1	0 to 10	0.075mm	0 to1

2.3.3 Edge Restraints

Edge restraints are an essential component of Interlocking concrete pavers, which are used at the interfaces with asphalt or concrete pavements. Edge restraints resist lateral movement, prevent rotation of the pavers under load and restrict loss of bedding sand material at the boundaries. Edge restraints are designed to remain stationary while receiving impacts during installation, from traffic loads and freeze-thaw cycles.

Edge restraints should be laid at all boundaries of the paved area or where there is change in pavement material. There are two general types of edge restraints. Those made elsewhere and installed at the site include precast concrete, plastic, cut stone, aluminum and steel. Restraints formed on-site are made of poured-in-place concrete. Some edge restraints like aluminum, steel require spiking to an aggregate base. Edge restraint should never be on top of the bedding sand.

Manufactures edge restraint like full depth concrete edging generally extends the depth of the base materials. Partial depth precast concrete may be used for residential and light duty commercial applications. Precast concrete and plastic edge restraints must be firmly anchored on a compacted aggregate base with steel spikes. Unlike plastic and precast edging, L-shaped aluminum or steel edging should never be placed on soil. They should be anchored in base course. Aluminum and steel edging can be used to provide smooth vertical surface against pavers and additional stability. Poured-in-place concrete edge restraint is the restraint formed on-site. It is suitable for pavers subjected to pedestrian traffic and for residential driveways and should extend well below the sand bedding layer (ICPI 2005).

2.3.4 Geotextile

Geotextile used as soil reinforcement are strong woven or knitted textiles made from polypropylene, polyester or glass fiber, or combination of these. On heavy traffic areas the soil reinforcement is placed as close to the pavement, here concrete blocks, as possible. The reinforcement is designed to increase the geotechnical bearing capacity and to increase the lifetime of the pavement (Stokkebo). Geotextile performs as a separation layer and they prevent soil from being pressed into base under loads, especially when saturated, thereby reducing the likelihood of rutting. The geotextile should not be placed on the top of the bedding sand (ICPI 2007).

2.3.5 Drainage

A network of perforated drain pipes in the open-graded base will be required to remove water since compaction will greatly reduce the soil's permeability (Burak 2007). Drainage pipes are installed perpendicular to the road surface at the lowest level of the concrete and asphalt bases before the construction of headers, curbs and bases. The drain pipes are filled with pea gravel and covered with geotextile to prevent the loss of bedding materials. The drainage pipes are essential in all the designs other than aggregate base pavement design.

2.3.6 Base and Subbase

One of the other major components of ICP is the base. When carefully constructed and regularly maintained interlocking concrete pavement should have long pavement life. The performance of this pavement depends upon the selection of type of base as well as construction (ICPI 2007). A wide range of materials are suitable for use as the base in ICP. The base layer can be constructed using untreated aggregate, asphalt concrete, cement stabilized granular materials and lean concrete. Comparisons have been made of the performance under traffic of ICP laid on various types of bases. For identical base and pavement thickness the best levels of performance are usually achieved by using cement-treated bases followed by the use granular base (Shackel 1980).

The layer directly below the base is known as subbase. Its purpose is to transfer traffic imposed loads from overlying structures to the supporting embankment. The subbase layer is usually consists of an unbound layer, although cement bound materials are sometimes used. In some situations, where the subgrade has a high load bearing capacity or where anticipated traffic loading is reasonably light the subbase may be omitted.

2.3.7 Subgrade

The layer of soil immediately beneath the aggregate base or subbase is subgrade. The infiltration capacity of the subgrade determines how much water can exfiltrate from the aggregate into the surrounding soils. ICP's are flexible pavements, and as such, rely on load distribution in the base and adequate support from the soil subgrade. The subgrade soil is generally not compacted. The strength of the subgrade is a key factor in the design of ICP. The characteristics and behaviour of subgrade soils have a major influence in the design and performance of flexible pavement systems. In general, the weaker the subgrade, the greater the thickness of pavement required. Many procedures for establishing this design factor are available: e.g., estimates made by the engineer based on experience, soil-type-to-strength correlations, laboratory tests, and in situ evaluation methods such as dynamic deflection tests. The method most widely used to characterize the bearing capacity of subgrade is the California Bearing Ratio (CBR) test.

Chapter 3

Crosswalk Construction

3.1 CPATT Test Track

3.1.1 Project Location

The first series of crosswalks is situated at the Centre for Pavement and Transportation Technology (CPATT) Test Track in the south east corner of the Regional Municipalities of Waterloo Waste Management Facility, located at 925 Erb Street West in the City of Waterloo. The University of Waterloo, CPATT Test Track is as shown in Figure 1.

There are four crosswalks with different bases and bedding materials located at the first section of the Test Track. The centreline of the first crosswalk is located at 0+060, the second is at 0+070, the third is at 0+080 and the fourth is at 0+090 of the Test Track as shown in Figure 1.

Figure 1 CPATT Test Track Satellite View

Four 8.4m long and 3m wide sections were cut on existing HL3 Asphalt pavement to build the test sections. The construction of test sites was started on June 18, 2007 and completed on June 26, 2007 for three crosswalks and the fourth crosswalk at the CPATT Test Track started on July 16, 2009 and completed on July 24, 2009. Interlocking concrete pavers were installed manually by a crew of two to three members. Several sensors were installed to measure the response of the pavement in various locations during construction.

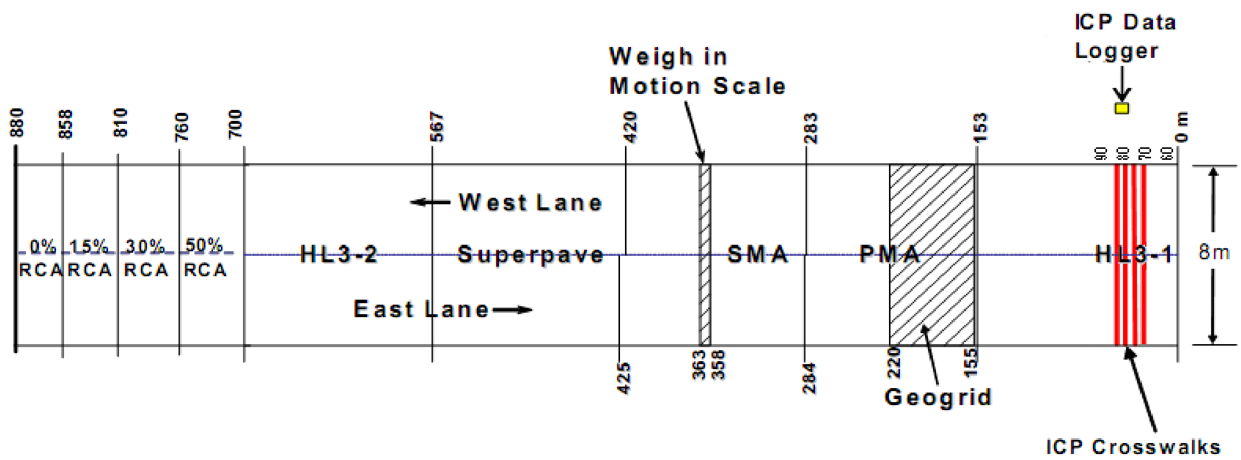


Figure 2 University of Waterloo, CPATT Test Track Layout

3.1.2 Preconstruction Phase

The preconstruction phase included site selection, layout, sensors design and preliminary sensors checking. The approximate location of the research project was finalized during the preconstruction survey by Susan Tighe, University of Waterloo, Rob Burak, ICPI, Ross Yantzi, Ross Yantzi Pavestone Plus Limited and Sudip Adhikari, University of Waterloo graduate student. The centreline of all three crosswalks and approximate location of the data logger were marked at this stage.

Four different type of sensors namely vibrating wire strain gauges, earth pressure cells, temperature profiles and moisture probes were placed to determine vertical strain in asphalt and concrete bases, vertical earth pressure in base and subbase and temperature and moisture variation at different elevation in subbase.

A preliminary check of the sensors was made by connecting to the readout box and observing the displayed readout prior to the installation.

3.1.3 Pavement Section

The proposed project is retrofitted on HL3 asphalt pavement. HL3 is a standard asphalt mix design for cities and municipalities. It is commonly used on local collector and arterial roads. The structural components of the existing pavement are shown in Figure 3.

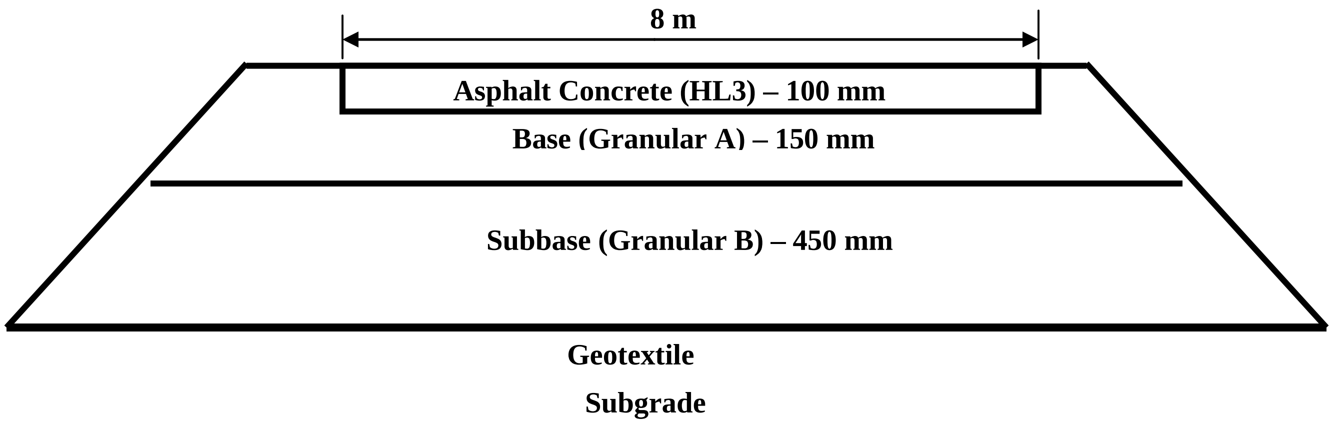


Figure 3 Existing Road Cross Section

A typical interlocking concrete paver crosswalk consists of concrete pavers placed on top of a layer of bedding sand over a base and subbase layers. Four different designs selected for this study are as follows and are shown in Figure 4, 5, 6, 7 and 8:

- Sand Set Granular Base Aluminum Header (SSGBAH)
- Sand Set Concrete Base Concrete Header (SSCBCH)
- Sand Set Asphalt Base Steel Header (SSABSH)
- Bituminous Set Concrete Base Concrete Header (BSCBCH)

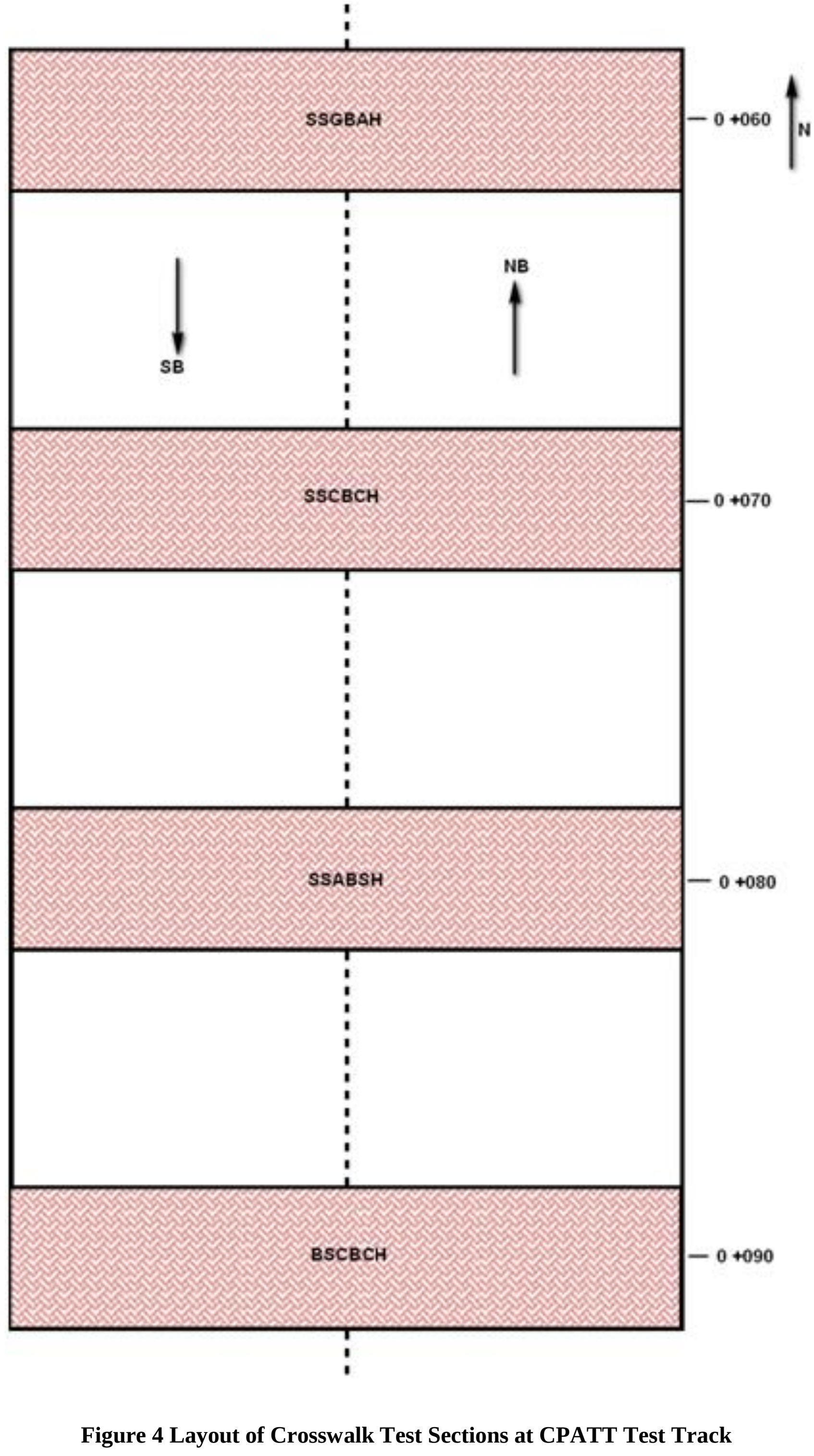


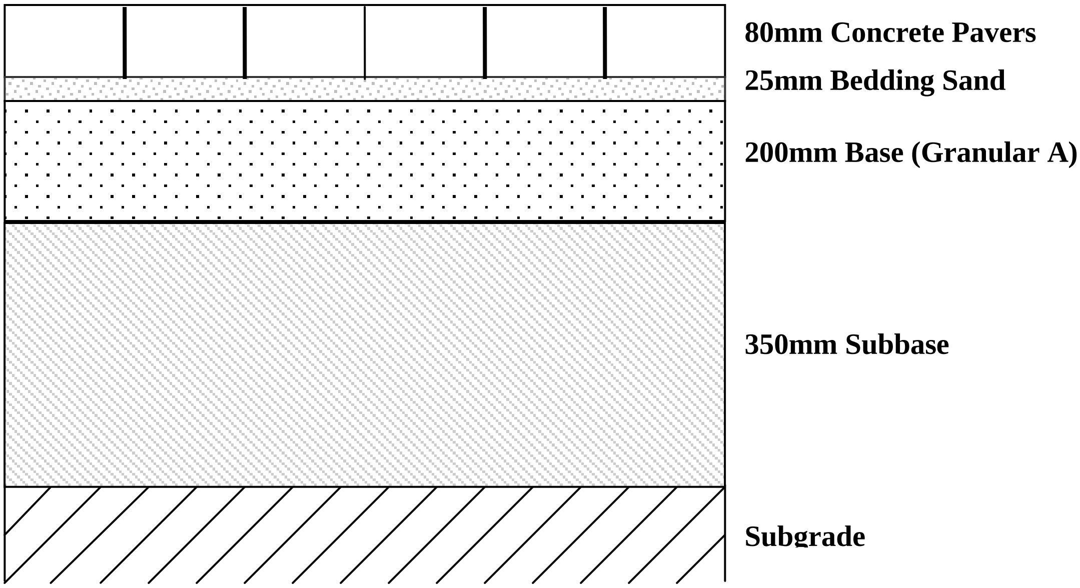
Figure 4 Layout of Crosswalk Test Sections at CPATT Test Track

The first crosswalk design is composed of 80mm interlocking concrete paver on the top of 25mm bedding sand layer, 200 mm thick granular base is built on the top of 350mm granular subbase, as shown in Figure 5 and is called Sand Set Granular Base Aluminum Header (SSGBAH). The L-shaped aluminum angle edge restraints are designed on top of the asphalt base on the both sides parallel to the center line of the crosswalk section.

The second crosswalk design is composed of 80mm interlocking concrete paver on the top of 25mm bedding sand layer, 200mm thick concrete base is built on the top of 400mm granular subbase as shown in Figure 6 and is called Sand Set Concrete Base Concrete Header (SSCBCH). The 150mm wide concrete header is placed and continues as a restraint on the transverse sides. Note that in a typical construction there would be a curb edge on each side but this is not the case at the Test Track so it was necessary to form a concrete restraint on the transverse ends of all of the Test Track sections.

The third crosswalk design is comprised of 80mm interlocking concrete paver on the top of 25mm bedding sand layer. The 100mm thick asphalt base is built on the top of 50mm granular A and 450 mm granular B subbase as shown in Figure 7 and is called Sand Set Asphalt Base Steel Header (SSABSH). Two concrete curb restraints along the edge of the road are parallel to the road centerline while the L-shaped iron angle edge restraint with dimension of 95mm x 95mm x 6mm are designed on top of the asphalt base on the both sides parallel to the center line of the crosswalk section.

The fourth crosswalk design is composed of 80mm interlocking concrete paver on the top of 25mm bituminous sand layer. Concrete base with 200mm thickness is built on the top of 400mm granular subbase as shown in Figure 8 and is called Bituminous Set Concrete Base Concrete Header (BSCBCH). The 150mm wide concrete header is placed and continues as a restraint on the transverse sides.



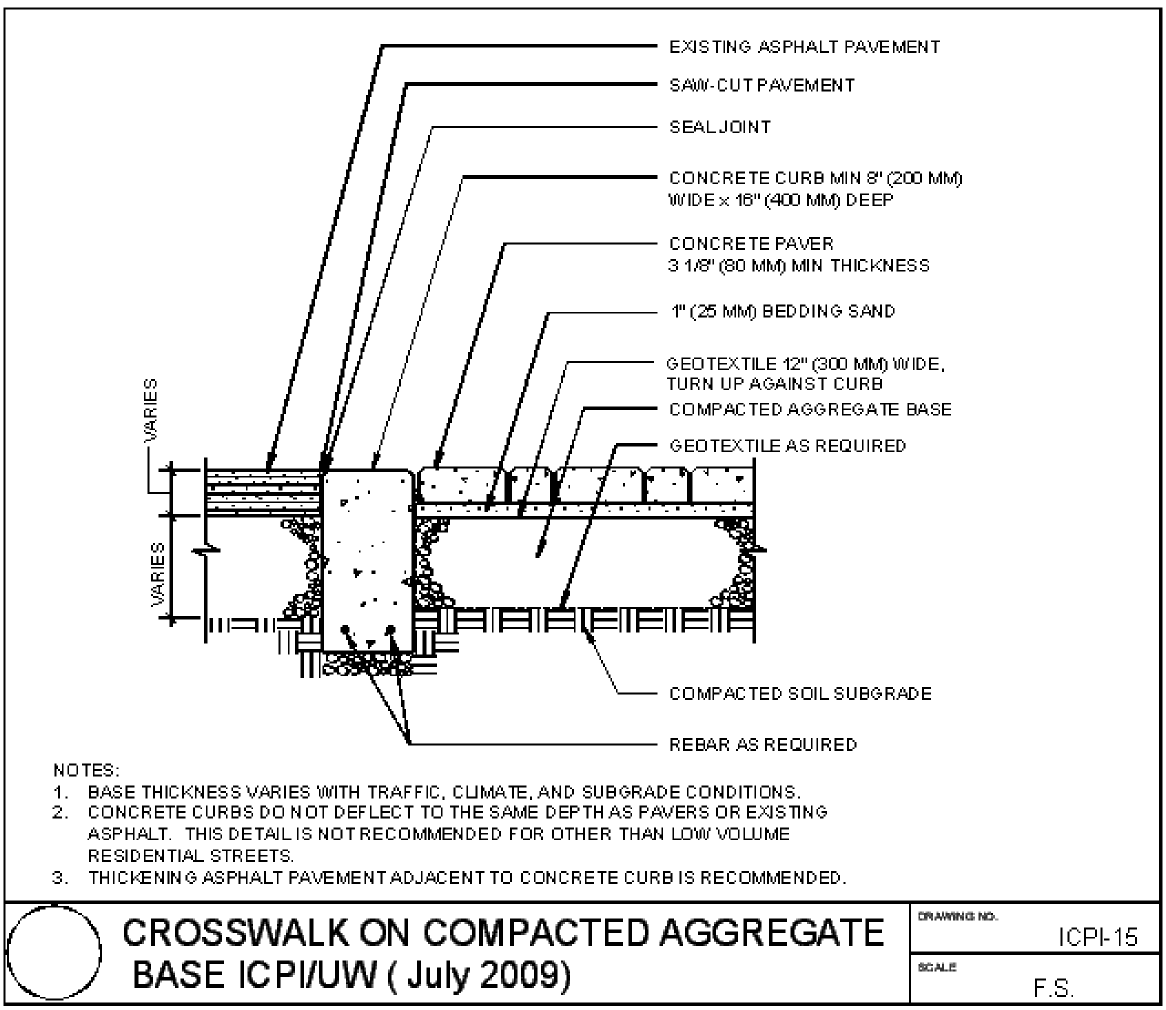
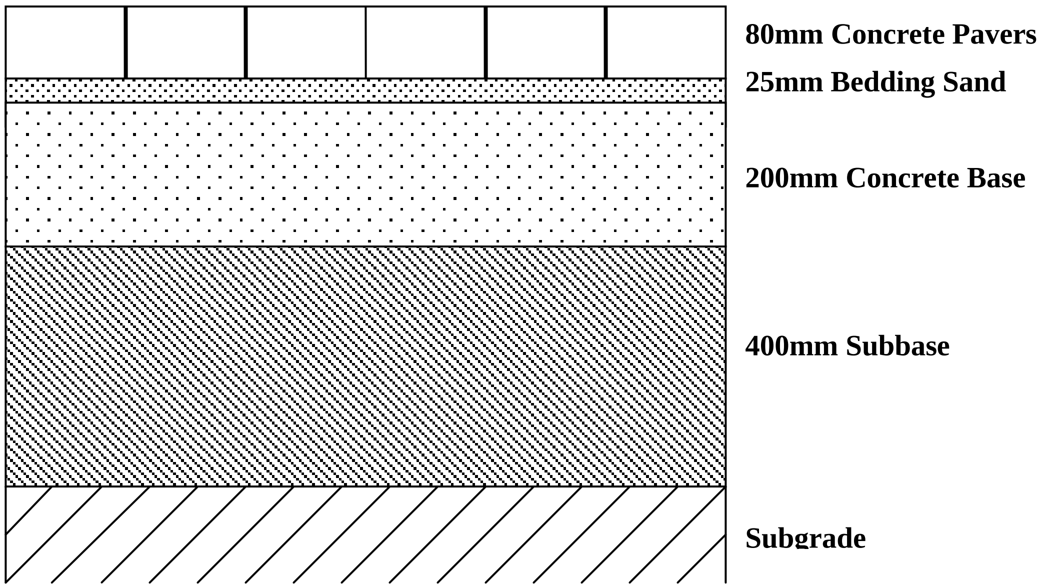


Figure 5 Sand Set Granular Base Aluminum Header (SSGBAH)



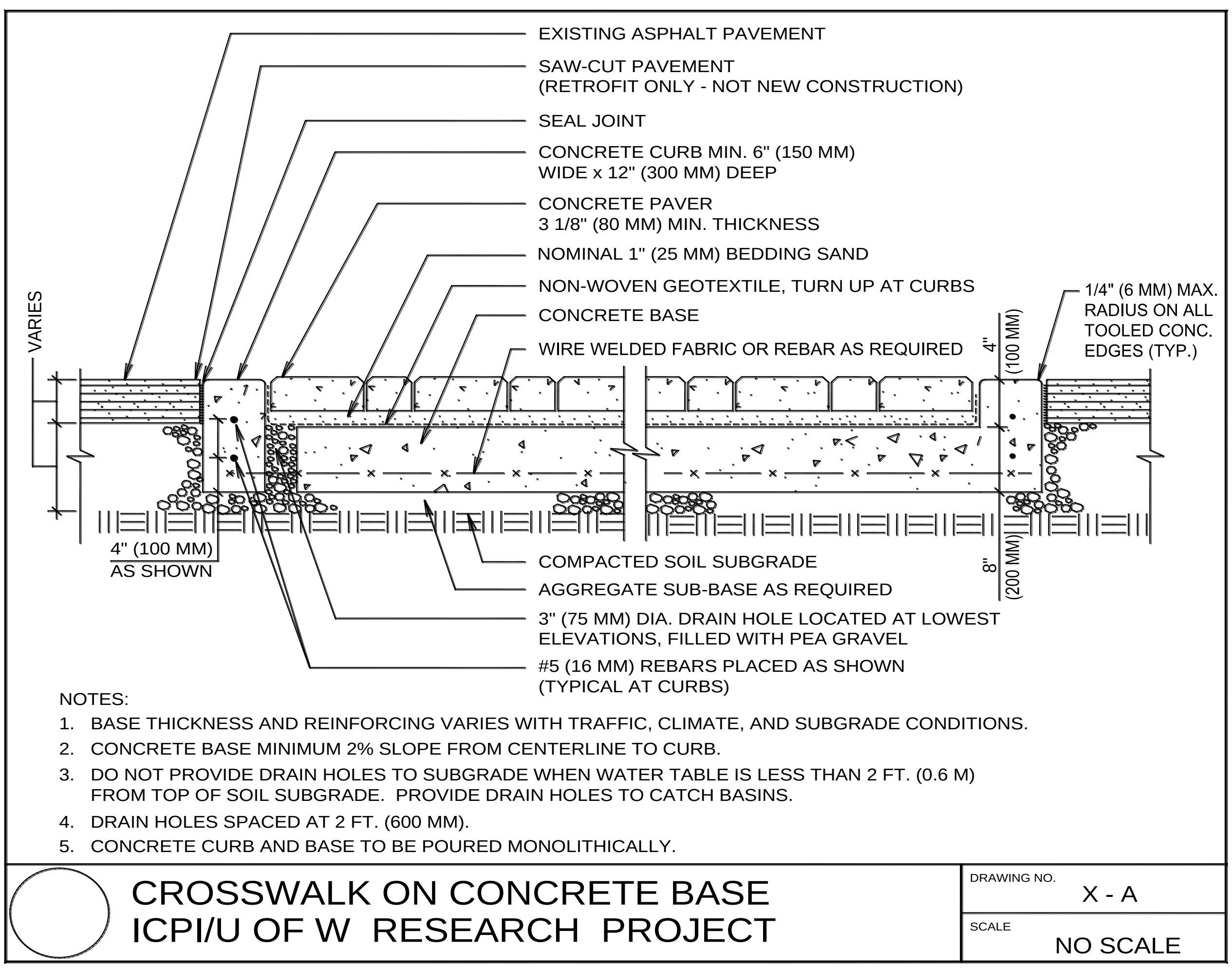
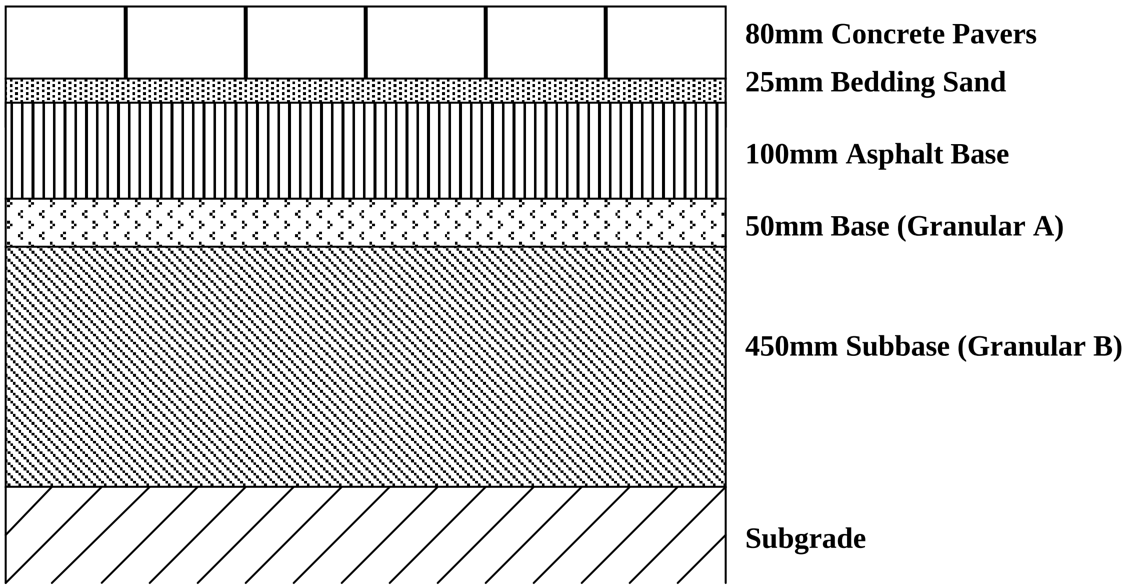


Figure 6 Sand Set Concrete Base Crosswalk Section (SSCBCH)



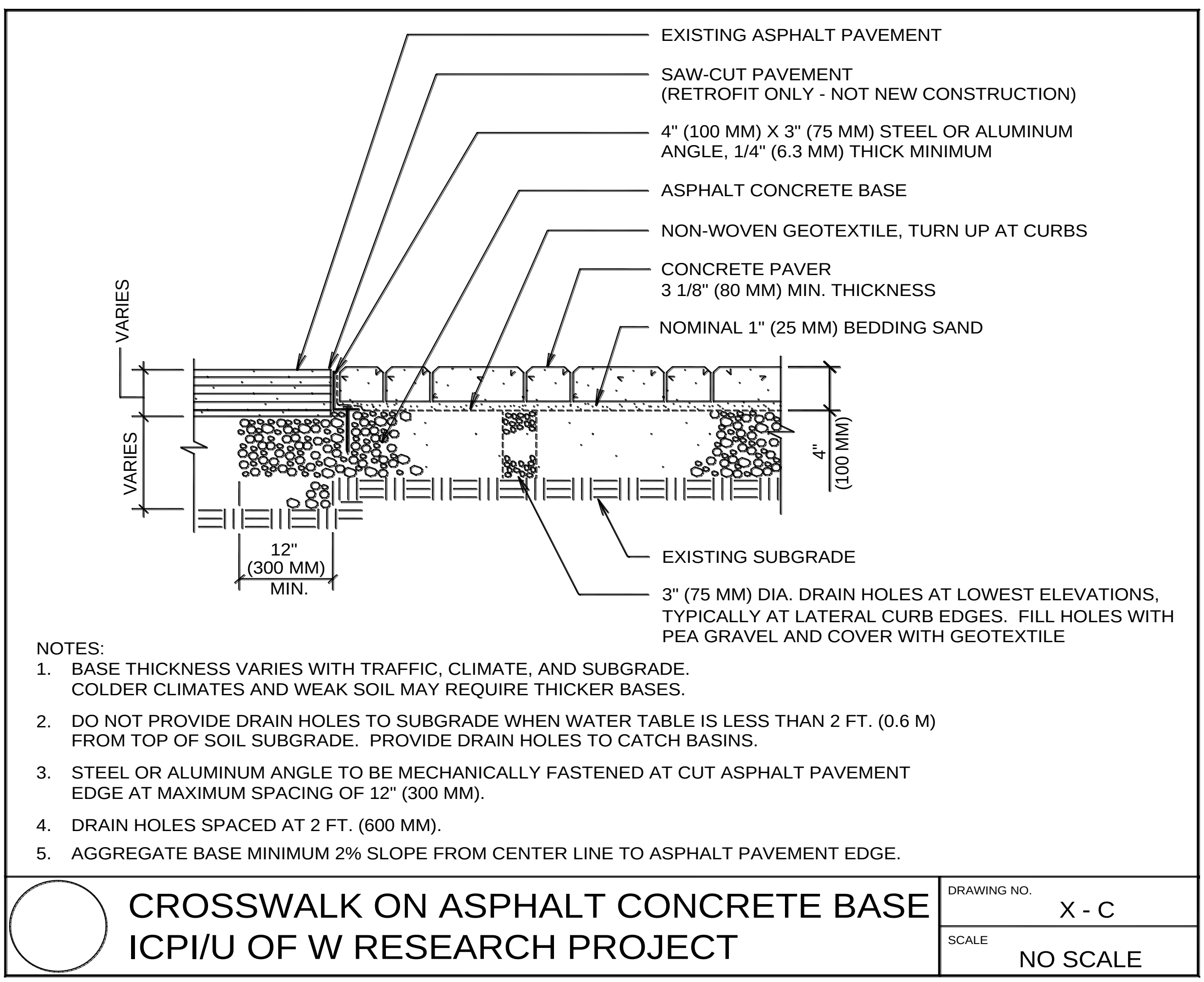
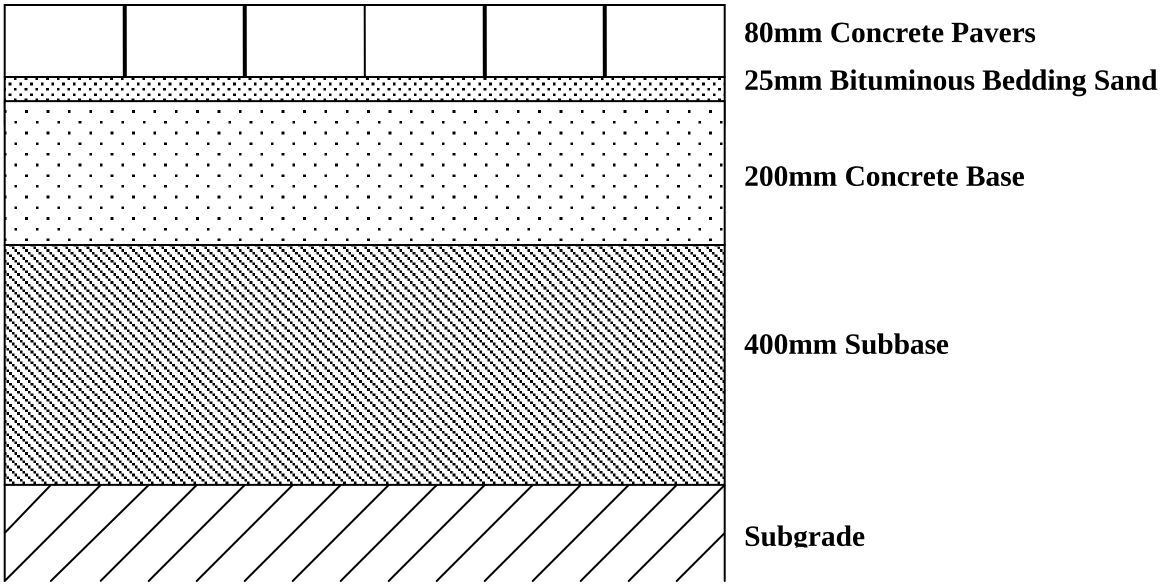


Figure 7 Sand Set Asphalt Base Crosswalk Section (SSABSH)



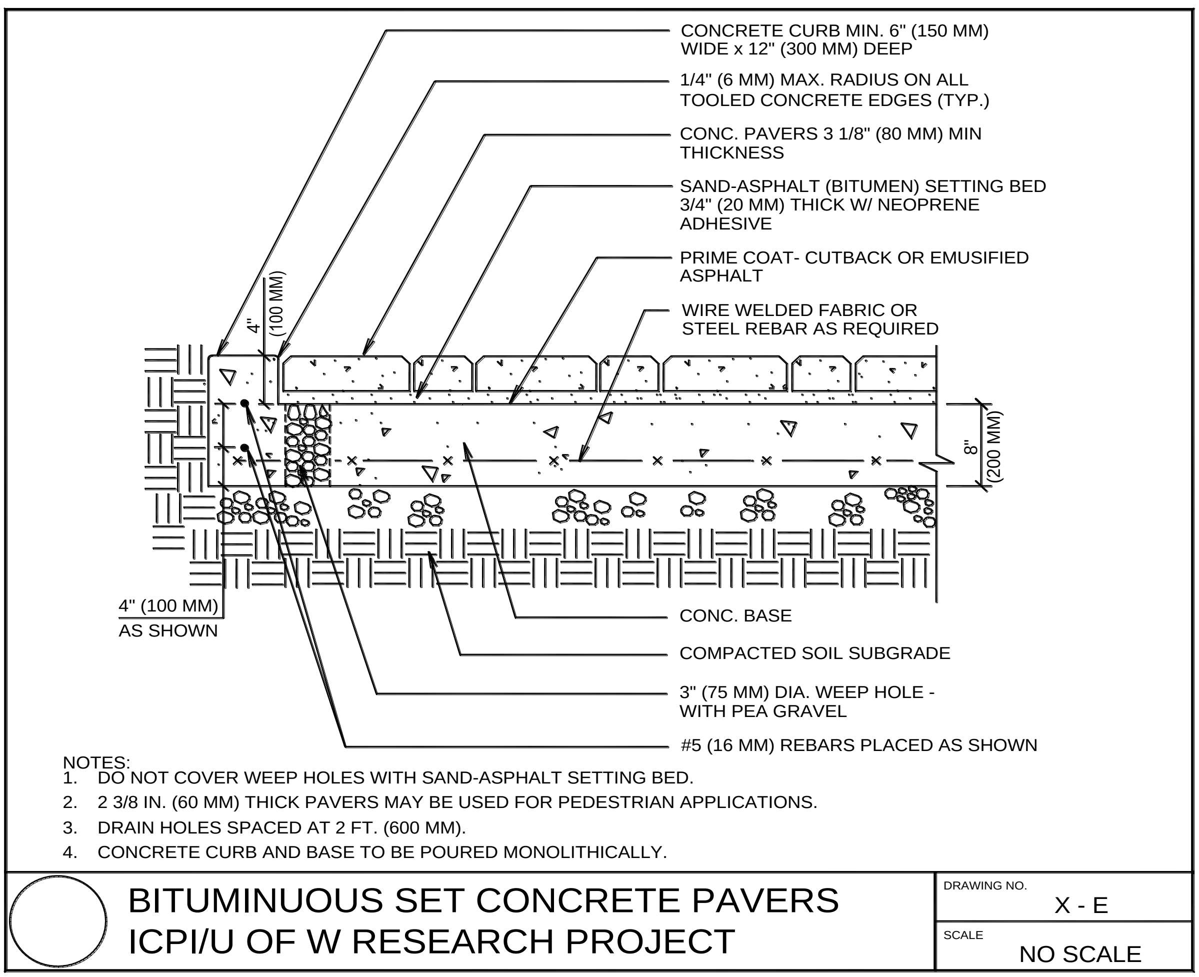


Figure 8 Bituminous Set Concrete Base Crosswalk Section (BSCBCH)

3.1.4 Construction Phase

The construction of the CPATT interlocking concrete crosswalks at the CPATT Test Track were carried out by Ross Yantzi's Pavestone Plus Limited in two phases as described below.

3.1.4.1 First Phase

The first phase of crosswalk construction was started on June 18, 2007 and completed on June 26, 2007 for three crosswalks SSCBCH, SSABSH and BSCBCH. The interlocking concrete pavers were supplied by NAVA STONE in Cambridge, Ontario. The entire construction phase lasted approximately seven days. Each of the crosswalks is 3m in width and 8.3m in length. The description of the activities is described in following sections.

3.1.4.1.1 Excavation of the Asphalt Pavement

The CPATT crosswalk construction involved the excavation of the existing asphalt section. The carpenter square was used to mark the section to be cut. The cutting of the sections started approximately at 8:30 a.m. on June 18, 2007 and the 100mm thick asphalt layer for all three sections was cut in approximately four hours as shown in Figure 9. Immediately following the cutting, the excavator removed the asphalt pavement and granular A materials to create a trench for each section with an approximate depth of 300mm for placing the crosswalk base. The entire excavation process took approximately five hours and was completed at 2 p.m. on June 18, 2007.

Figure 9 Excavation of Asphalt Pavement

3.1.4.1.2 Crosswalk Base Construction

Within the three crosswalk test sections, SSCBCH and BSCBCH have a concrete base underneath the ICP while SSABSH has an asphalt base underneath the ICP. The southernmost crosswalk BSCBCH base was constructed first. After excavating the trench, the sensors designed to be placed in the subbase were installed. The subbase course then was levelled and compacted with a 20 KN/ 4500 lbs vibrating plate compactor. Wooden forms were installed around the perimeter of the crosswalk for curbs and headers. Reinforcement rebars having mesh size of 150mm /150mm were placed on the top of the subbase course before the concrete pouring as shown in Figure 10. High Early concrete from Dufferin Concrete's Forwell Plant in Kitchener was brought to the construction site at 1:20 PM on June 20, 2007 and poured into the forms from 1:30 PM to 3:35 pm. A total of 6.5m^3 of concrete poured in place over a two hour period. The first concrete base crosswalk was constructed on June 21, 2007 using concrete as the third crosswalk's base. The concrete pouring started at 1:10 PM and lasted one and half hours as shown in Figure 10.

Figure 10 Concrete Base Construction

The length of the concrete base is 8.3m including curbs and the width is 3m including the headers on both sides. The thickness of the concrete bases is 200mm. Each base has curb and headers around the perimeter; the width of the curb/header is 150mm. Exposed concrete headers edges are trimmed to 3mm radius to reduce the likelihood of chipping. Three control joints for shrinkage were provided at three locations throughout the base. The procedure is shown in Figure 11, 12 and 13.

The construction of the asphalt base crosswalk was started on June 22 at 9.30 am and took approximately two hours to complete the first 50mm lift. Within the two hours of paving, 3.6 tonnes of HL3 was spread, screeded, and compacted with a 20 kN plate compactor as shown in Figure 13. The paving of the second/final 50mm lift of asphalt was started on the same day at 2.00 pm and completed at 4.00 pm. The air temperature during this operation ranged from 27 to 34 degree Celsius. The mix temperature on arrival to the site was 114°C and 106°C before compaction of the first lift. Similarly, the mix temperature of the second lift before the compaction was 72°C. The mix was prepared at Kitchener Asphalt Plant of Steed and Evans Limited.

The asphalt base also has two concrete curbs similar to ones for the concrete bases along its two edges that are parallel to the Test Track and these curbs were constructed using the remaining concrete after constructing the second concrete base crosswalk.

Prior to the concrete pouring of the base, eight ABS pipes with diameter of 75mm and length of 300 mm were installed perpendicular to the road surface at the lowest level of the base course. The pipes were filled with pea gravel and covered with geotextile to prevent the loss of bedding materials. Their purpose is to remove excess water from the bedding sand. Since the road base is constructed with two percent slope towards the edge, four pipes were installed along the two outer edges of the bases. One set of four drainage pipes are located in the northbound lane and another set are located in the southbound lane. The distance between adjacent drainage pipes (centre to centre) is approximately 80cm and the first drainage pipe of each set is 20cm offset from the edge of the base.

Figure 11 Formworks, Drain Pipes and Reinforcement Bars for Concrete Base Section

Figure 12 Concrete Placement

Figure 13 Concrete Base Section

Figure 14 Asphalt Base Section

3.1.4.1.3 Bedding Material Placement

Prior to placing the interlocking concrete pavers, a layer of bedding material was placed in the base of each section. The CSA requirements and actual gradation of the bedding sand is provided in Table 3. Micro-Deval test was also performed before the placement as per ICPI Tech Spec Number 17. An 8.9 % Micro-Deval degradation loss was calculated which is slightly greater than maximum recommended value 8 %.

Table 3 Gradation of Bedding Sand used in the field

(CSA Requirements)

Sieve Size (mm)	Percent Passing (%)
10 mm	100
5 mm	95-100
2.5 mm	80-100
1.25 mm	50-90
0.630 mm	25-65
0.315 mm	10-35
0.160 mm	2-10
0.075 mm	0-1

(Actual)

Sieve Size (mm)	Percent Passing (%)
9.5 mm	100
4.75 mm	99.9
2.36 mm	86.6
1.18 mm	71.1
0.600 mm	53.2
0.300 mm	26.3
0.150 mm	6.2
0.75 mm	1.1

For SSCBCH, a layer of non-woven geotextile was placed in the constructed base and a layer of bedding sand with approximate thickness of 25mm was spread and screeded on top of the non-woven geotextile as shown in Figure 15.

For SSABSH, L-shaped iron angle were installed on top of the asphalt base and along the cut pavement edge. The angle iron is 9.5cm wide and 9.5cm high and 6mm thick. Nails and screws were drilled through the angle iron and into the asphalt base in every 60cm to fasten the angle iron onto the base. On top of the angle iron and the asphalt base, a similar layer of non-woven geotextile and bedding sand as SSCBCH were placed.

For BSCBCH, the concrete base surface was prepared with an emulsified asphalt tack coat. A hot-sand asphalt mix was brought to the site and spread and compacted to 25mm thick layers. After the asphalt cooled, a thin coating of asphalt-neoprene adhesive was applied across the surface.

Figure 15 Bedding Sand and Non-woven Geotextile

3.1.4.1.4 Placement of Interlocking Concrete Pavers (ICP)

The interlocking concrete pavers were placed into the concrete/asphalt base at SSCBCH, SSABSH and BSCBCH on June 22, 26 and 25, respectively. Approximately 1070 pieces of interlocking concrete pavers with approximate dimension of 200mm × 100mm × 80mm were placed in each section. The pavers were laid in 45° herringbone pattern. Installation was started from the corner with securing 45° string lines on the bedding course. Edge pavers are saw cut to fit against the sailor and soldier courses. The sailor and soldier are interlocking concrete pavers which were placed in such a way that SSABSH and BSCBCH have three sides (one long side and two short sides) with sailor course and one long side with soldier course. In contrast, SSCBCH has sailor courses on both sides. After the installation, the surface was compacted with a vibratory plate to compact the bedding sand, seat the pavers in it and force the bedding sand into the joints at the bottom of the pavers. Figure 16 provides details of the paver while Figure 17 and 18 presents the layout. The final elevation of the surface course was kept 6mm above the adjacent asphalt pavement to accommodate any future settlement.

<u>Physical Characteristics of the Pavers</u>

Brand – Cobble100-80^{TM}

Color – Antique Red (Red/

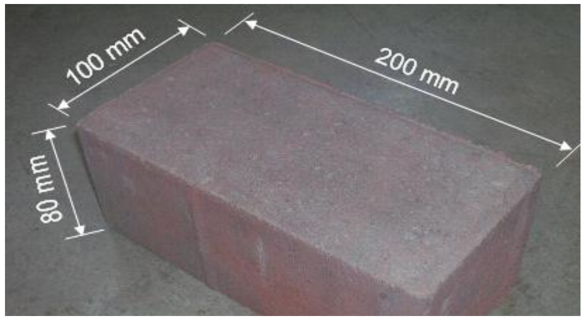


Figure 16 Physical Characteristic of the Concrete Pavers

Figure 17 Laying Interlocking Concrete Pavers

Figure 18 Laying Interlocking Concrete Pavers

3.1.4.1.5 Joint Sands Placement and Compaction

After initial compaction of the pavers, dry joint sand was spread on the surface and compacted with a vibratory plate compactor to ensure that the spaces between pavers are filled. Excess joint sand was then removed.

3.1.4.2 Second Phase

The crosswalk SSGBCH was being researched in the Ring Road test site with city traffic but it was not constructed at Test Track. The performance of this crosswalk under heavy load was still a mystery. For this reason, SSGBCH was constructed in the Test Track research site on July 2009.

The second phase of crosswalk construction was started on July 16, 2009 and completed on July 24, 2009. SSGBAH was constructed in this period. The interlocking concrete pavers used in this crosswalk were from Permacon. The crosswalk is 3m in width and 8.3m in length.

3.1.4.2.1 Construction of Curb

On July 16, 2009, the concrete curb was constructed for the new crosswalk along the edges. The work started at 8 am with saw cut of asphalt for the curb construction followed by excavation of asphalt, construction of form work and concrete placement on both sides.

Figure 19 Construction of Curb

Figure 19 shows the construction of the curb on the southbound lane. The construction of the curb occurred one week in advance of the construction to allow for proper curing. The curb area was protected by pylons and reopened to traffic. The saw cut of asphalt for the whole crosswalk section was completed the same day to decrease the work load on July 22. The work was completed around 3:30 pm.

3.1.4.2.2 Excavation of Asphalt Pavement and Existing Granular ‘A’

Since the saw cut of asphalt section was already completed, the work started at 8:15 am on July 22nd by excavating the asphalt layer in the southbound lane. Following the cutting of the asphalt sections, the excavator removed existing road base material (Granular ‘A’). The leveling was completed during this process with the assistance of the laser level to ensure an accurate depth to be excavated. Approximately 300mm of depth was maintained. The process was completed in two hours. Figure 20 shows the excavation of existing asphalt pavement in the southbound lane. And Figure 21 shows the excavation of Granular A with the laser level on site.

Figure 20 Excavation of Asphalt Pavement

Figure 21 Excavation of Granular A

The moisture sensors and temperature sensors were installed by the CPATT team after the excavation of base material in the designed location were completed. The work started at 10:50am and the installation process took almost three hours. The details of installation of sensors are described in the Instrumentation section of this report.

3.1.4.2.3 Crosswalk Base Construction

After the completion of sensor installation by the CPATT group on the subbase, at 2:45 pm, the subbase was compacted with a vibrating plate compactor. Caution was taken around the sensors. Crosswalk base material which is granular 'A' in this case was placed up to the height of 200 mm above subbase.

The base material was then compacted with 1000 lbs vibrating plate compacter. The vibratory plate compactor repeated six times to achieve 98% of compaction. The corners and the sides of the base were compacted with a Jumping Jack compactor with 254mm plate on the bottom. Figure 22 shows the vibratory plate compactor in operation in the northbound lane. The photo shows a group of students who also came to visit the site to see the construction work.

Figure 22 Vibratory Plate Compactor in Crosswalk Base

3.1.4.2.4 Installation of Header and Geotextile

On Day 2 of construction, July 23, 2009 the work started around 9:15 am but, was delayed for an hour because of rain. Granular 'A' was again compacted in the morning to confirm compaction with a small compactor and levelling was done for the base material. Aluminum header was brought from Permaloc Corp. The header was fastened in the granular base with 254mm long steel spikes as shown in Figure 23. The nails were spaced approximately 200 mm which were from Tree Island Industries. After the successful installation of Header, non-woven geotextile was spreaded over the crosswalk base.

Figure 23 Installation of Aluminum Header

3.1.4.2.5 Bedding Material Placement

The main function of bedding sand is to provide support to the pavers during installation and it is also an important structural component. It also helps to maximize interlock among pavers, and facilitates drainage of water that infiltrates through the joints. In this research project, the bedding sand materials were transported from Barrie, Ontario and were of good quality. The Gradation test for the sand recommended by ICPI, according to ASTM C 33 was performed in the CPATT laboratory. Table 4 summarizes the ASTM requirements and actual gradation of sand.

Table 4 Gradation of Bedding Sand for SSGBCH construction

Sieve Size	Percent Passing	Actual Measured
3/8 inch	100	100
No. 4	95-100	97.8
No.8	85-100	84.5
No.16	50-85	64
No.30	25-60	47.9
No.50	10-30	26.7
No.100	2-10	9.5
No.200	0-1	1

The Micro-deval test was also carried out in the CPATT laboratory on the samples and the result confirms the ICPI Tech Spec 17. Micro-deval gradation loss was calculated to be 8.32% which is only slightly higher than the 8% recommended by ICPI. The test was carried out through ASTM specification.

The 25mm of bedding sand was spread above the non-woven geotextile by placing screed rails on it. The bedding sand was screeded across the rails with a straight strike board as shown in Figure 24. Screed rails were removed after that, filling the void with the bedding sand.

Figure 24 Spreading and Screeding of Bedding Sand

3.1.4.2.6 Placing Interlocking Concrete Pavers

On Day 2 of construction, July 23, 2009 the Interlocking Concrete Pavers were placed on the crosswalk. Concrete pavers were placed in 45 degree herringbone patterns as shown in Figure 25. The interlocking concrete pavers used in the new crosswalk were from Permacon with a dimension of 200mm X 100 mm X 80 mm. The Herringbone pattern is recommended in ICPI Tech Spec 2 as these interlocking patterns provide the maximum load bearing support (3). Construction was started from the corner with utmost care in the joint width, which should be between 2 mm to 5 mm according to Tech Spec 2. The pavers on the edges were saw-cut to maintain the pattern and cut pavers were used to fill gaps along the edges of the pavement.

Figure 25 Interlocking Concrete Pavers

After the installation of the paver was completed, it was compacted with a vibrating plate compactor. Approximately three passes were made across the pavers to seat the pavers in the bedding sand. Since the construction was retrofitted to existing asphalt pavement, future settlement was considered and the elevation of the finished crosswalk was approximately 5mm above the adjacent asphalt pavement.

3.1.4.2.7 Joint Sand Placement and Compaction

Joint sand should be finer than the bedding sand to facilitate filling of the joints. Bedding sand can also be used to fill the joints. In this research project, during the construction the HP Polymeric Joint Sand from Techni-Seal was used. The sand was spread over the completed crosswalk and was swept out into the joints. After sand is uniformly spread, the pavers were compacted again using a vibratory plate compactor. However, after final completion of crosswalk, bedding sand was spread and compacted with a compactor as shown in Figure 26.

Figure 26 Spreading of Bedding Sand on the Pavers

3.1.5 Instrumentation

Pavement instrumentation is crucial to understanding material performance in the field, as well as pavement system response to loading and environment. Installation of sensors in the pavement allows the CPATT research team and ICPI to better understand the behaviour of interlocking concrete pavement performance during freeze thaw conditions. The goal of instrumentation is to assess the in-situ performance related to stress, strain, temperature and moisture. Sensors installed in pavement sections are divided into two categories. Temperature and TDR probes are installed to measure environmental responses whereas pressure cells and strain gauges are embedded to capture loading responses.

The sensors were installed during construction of the project**.** Different types of sensors namely vibrating wire strain gauges, earth pressure cells, temperature profiles/probes and moisture probes/sensor were placed to determine vertical strain in asphalt and concrete bases, vertical earth pressure in base and subbase and temperature and moisture variation at different elevation in subbase. The sensors were installed at different locations inside the pavement and all cables were collected at one point and placed in a trench together and routed to the edge. Phase 1 and phase 2 have two separate conduits running parallel to each other to the Data logger. Table 5 summarizes the purpose and locations of the sensors.

Table 5 Summary of Instrumentation

Crosswalk	Sensor	Quantity	Location	Purpose
(SSGBAH)	Temperature Probes	4	100mm and 300mm below from the top of the subbase at 1.5m offset from road edge	Measure the change of temperature
	Moisture Sensors	2	200mm and 375mm below from the top of the subbase at 3m offset from road edge	Measure in-situ moisture content
(SSCBCH)	Concrete Strain Gauge (VWSGE)	2	50mm and 150mm below from top of concrete base at 1.15m offset from road edge	Measure change of strain in concrete
	Earth Pressure Cells (LPTPCO9-V)	2	Bottom of concrete base and at 250mm below from bottom of base at 1.6m offset from road edge	Measure vertical stress in base and subbase
	Thermistor (TH0003-250-2)	2	50mm and 250mm below from bottom of concrete base at 1.4m offset from road edge	Measure the change of temperature
	Moisture Probes (6005L40WGL60 60 cm)	2	100mm and 250mm below from bottom of concrete base at 3m offset from road edge	Measure in-situ moisture content
(SSABSH)	Asphalt Strain Gauge (VWSGEA)	3	Bottom of the asphalt base at 1.1m, 2.2m and 3.3m offset from road edge	Measure the change of strain in the asphalt
	Earth Pressure Cells (LPTPCO9-V)	2	Bottom of concrete base and at 250mm below from bottom of base at 1.6m offset from road edge	Measure vertical stress in base and subbase
	Thermistor (TH0003-250-2)	2	50mm and 250mm above from bottom of concrete base at 1.4m offset from road edge	Measure the change of temperature
	Moisture Probes (6005L40WGL60 60 cm)	2	100mm and 250mm above from bottom of concrete base at 3m offset from road edge	Measure in-situ moisture content
(BSCBCH)	Concrete Strain Gauge (VWSGE)	2	50mm and 150mm below from top of concrete base at 1.15m offset from road edge	Measure change of strain in concrete
	Earth Pressure Cells (LPTPCO9-V)	2	Bottom of concrete base and at 250mm below from bottom of base at 1.6m offset from road edge	Measure vertical stress in base and subbase
	Thermistor (TH0003-250-2)	2	50mm and 250mm above from bottom of concrete base at 1.4m offset from road edge	Measure the change of temperature
	Moisture Probes (6005L40WGL60 60 cm)	2	100mm and 250mm above from bottom of concrete base at 3m offset from road edge	Measure in-situ moisture content

3.1.5.1 Moisture Sensor

Two different kinds of moisture sensors are used in this research project as shown in Figure 27 and 28. A moisture sensor is a device which measures the in-situ moisture content. In the first construction phase a moisture sensor consisting of three 600mm long stainless steel rods for measuring moisture content was used as shown in Figure 27. Two probes are installed horizontally in each section at 100mm and at 250mm from the top of the subbase and at 3m offset from the road edge. In the second phase, moisture sensors which are as shown in Figure 28 were installed at depths of 200mm and 375mm from the top of the subbase in second phase of the project. The Figure shows the calibrated sensors ready to be installed in the field. The offset from the road edge was three meter.

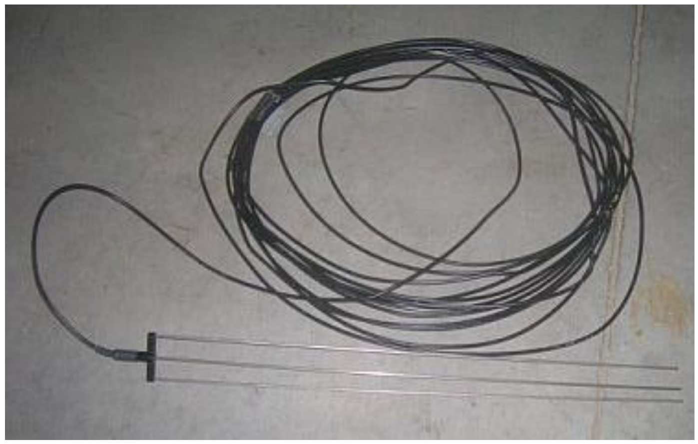


Figure 27 Moisture Probes for SSABAH, SSCBCH, and BSCBCH

Figure 28 Moisture Sensors Ready to Install in SSGBAH

To install a moisture probe, a cavity of 700mm x 200mm x 300mm is excavated into the subbase to accommodate the moisture probe. A cable trench of 200mm wide x 250mm deep running from the cavity to the edge of pavement was also excavated as shown in Figure 29. This trench is used to run the cables of pressure cells and temperature probe. All sharp stone fragments were removed from the cavity and the trench. The cavity and the trench were filled with 5mm sand layer and the probes and cables were placed.

After placing the sensors and cables the trench was filled with sand and subbase materials and compacted with a Marshall hammer to ensure the density. Similarly, the second probe was installed at 150mm above the first probe and filled with sand and compacted with a Marshall hammer.

Figure 29 Installation of Moisture Probes in First Phase

In the second phase, to install the moisture sensor, a pit of 400mm depth is excavated. The sensors fitted in the sensor tree at the depths 375mm and 200mm from the top of the subbase were installed. Sensor tree was constructed on site to hold two sensors at various depths in same area. Figure 30 shows the sensor tree ready to be placed on site.

Figure 30 Moisture Sensors in Sensor Tree in Second Phase

3.1.5.2 Earth Pressure Cells

Earth Pressure Cells, a device to measure the vertical stress are constructed from two circular stainless steel plates and welded together around their periphery. An annulus exists between the plates, which is filled with de-aired glycol. The cell is connected via a stainless tube to a transducer forming a closed hydraulic system. As stress is exerted on the surface of the cell, it pressurizes the fluid within the cell, which in turn is measured by the pressure transducer.

Two pressure cells are installed horizontally in each section at the bottom of the concrete/asphalt base and at 250mm from the top of the subbase at 1.6m offset from the road edge.

To install pressure cells, a cavity of 700mm x 300mm x 300mm is excavated into the subbase to accommodate the pressure cells. Cable trench excavated for moisture probes cables is used to run the pressure cells cables as well. All sharp stone fragments were removed from the cavity and the trench. The cavity and the trench were filled with 5mm sand layer and the probes and cables were placed. After placing the pressure cells and cables, the trench was filled with sand and subbase materials and compacted with a Marshall hammer to ensure the density. Similarly, the second cell is installed at 250mm above the first cell and filled with sand and compacted with a Marshall hammer.

The primary function of pressure cells is to monitor the change in the stress-state of the overlying layers and to measure the increase in vertical pressure due to traffic loading. The following equation is used to calculate the change of vertical stress-state.

$$\sigma = E*(P_n-P_0)*10^{-6} \quad \text{(Equation 1)}$$

Where, σ = Vertical stress (MPa)

E = Elastic Modulus of material where the pressure cell is placed (MPa)

P_n= Current pressure reading

P_0 = Initial pressure reading

A negative value indicates compressive stress and a positive value indicates tensile stress in above equation. Besides stress, these gauges can also measure temperature.

Figure 31 Earth Pressure Cells Installation

3.1.5.3 Temperature Probe

Temperature probes are installed to measure temperature variation at different elevations within the pavement structure. The temperature probe consists of two thermistors at the distance of 200mm are installed vertically in the subbase in each section. For the first phase of the project, the thermistors are located at 50mm and 250mm from the top of the subbase at 1.4m offset from the road edge. Whereas in the second phase there are two each at 100mm and 300mm from the top of the subbase at 1.5m offset from the road edge.

To install the temperature probes, a cavity of 300mm x 300mm x 600mm is excavated into the subbase as shown in Figure 32. All sharp stones fragments were removed from the cavity and the trench. The cavity and the trench were filled with 5mm sand layer and the probes and cables were placed. After placing the temperature probe and the cables, the trench was filled with sand and subbase materials and compacted with a marshal hammer to ensure the density.

Figure 32 Temperature Probe Installation in First Phase

The temperature probes used in the second phase of the project is shown in Figure 33 and the Sensor tree which was constructed for installation is shown in Figure 34.

Figure 33 Temperature Probes

Figure 34 Temperature Probes in Sensor Tree

3.1.5.4 Vibrating Wire Strain Gauge

The strain gauges are designed to measure the change of strain and the change of temperature in the concrete. Concrete strain gauges (Model-VWSGE) are installed at 50mm and 150mm below from the top of the concrete base at the right wheel pathway at Test Track site. Two types of strain gauges are installed to measure a strain in asphalt and concrete bases as shown in Figure 35 and 36 respectively. Three asphalt strain gauges are placed at the bottom of the asphalt base at 1.1m, 2.2m and 3.3m offsets from the road edge. Alternatively, concrete strain gauges are installed at the depth of 50mm and 150mm below from the top of the concrete base at 1.15m offset from the road edge.

Figure 35 Vibrating Wire Asphalt Strain Gauge

Figure 36 Vibrating Wire Concrete Strain Gauge

A HMA pad was placed at the asphalt strain gauges locations. After the asphalt is cooled, the strain gauges are hand placed and gently pressed into the mix as shown in Figure 37. A shallow cable trench was excavated and routed the cables to the edge. The backfill then was filled with sand and compacted with the Marshall hammer.

Figure 37 Asphalt Strain Gauge Installation

Concrete strain gauges are attached with U-shaped chairs and the chairs were driven into the ground and tied together to prevent from moving as shown in Figure 38. A shallow cable trench was excavated and routed the cables to the edge. The backfill then was filled with sand and compacted with the marshal hammer.

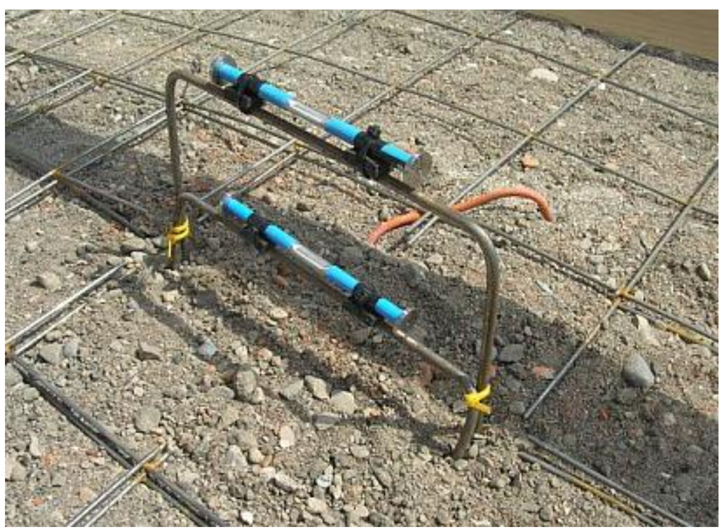


Figure 38 Vibrating Wire Concrete Strain Gauges Installation

The strain gauge can measure actual strain changes due to changes in moisture content of the concrete and stresses from traffic loading. Thermal correction factor is used to adjust change in strain due to temperature changes. The following equation is used to convert measured resonance reading into change of strain.

$$\varepsilon_t = (S_n - S_0) * GF + (T_n - T_0) * (TC_{c/a} - TCs) \quad \text{(Equation 2)}$$

Where, ε_t = True strain (microstrain)

S_n = Current resonance reading

S_0 = Initial resonance reading

GF = 1, Gauge factor for strain gauges model VWSGEA and VWSGE

T_n= Current temperature reading (°C)

T_0= Initial temperature reading (°C)

TCc/a= Thermal Coefficient of Concrete/Asphalt,

TC_s=Thermal Coefficient of Steel

A negative value indicates compressive strain and a positive value indicates tensile strain in above equation.

Profile views of each type of crosswalk showing the various locations of sensors are presented in Figures 39, 40, 41 and 42 respectively.

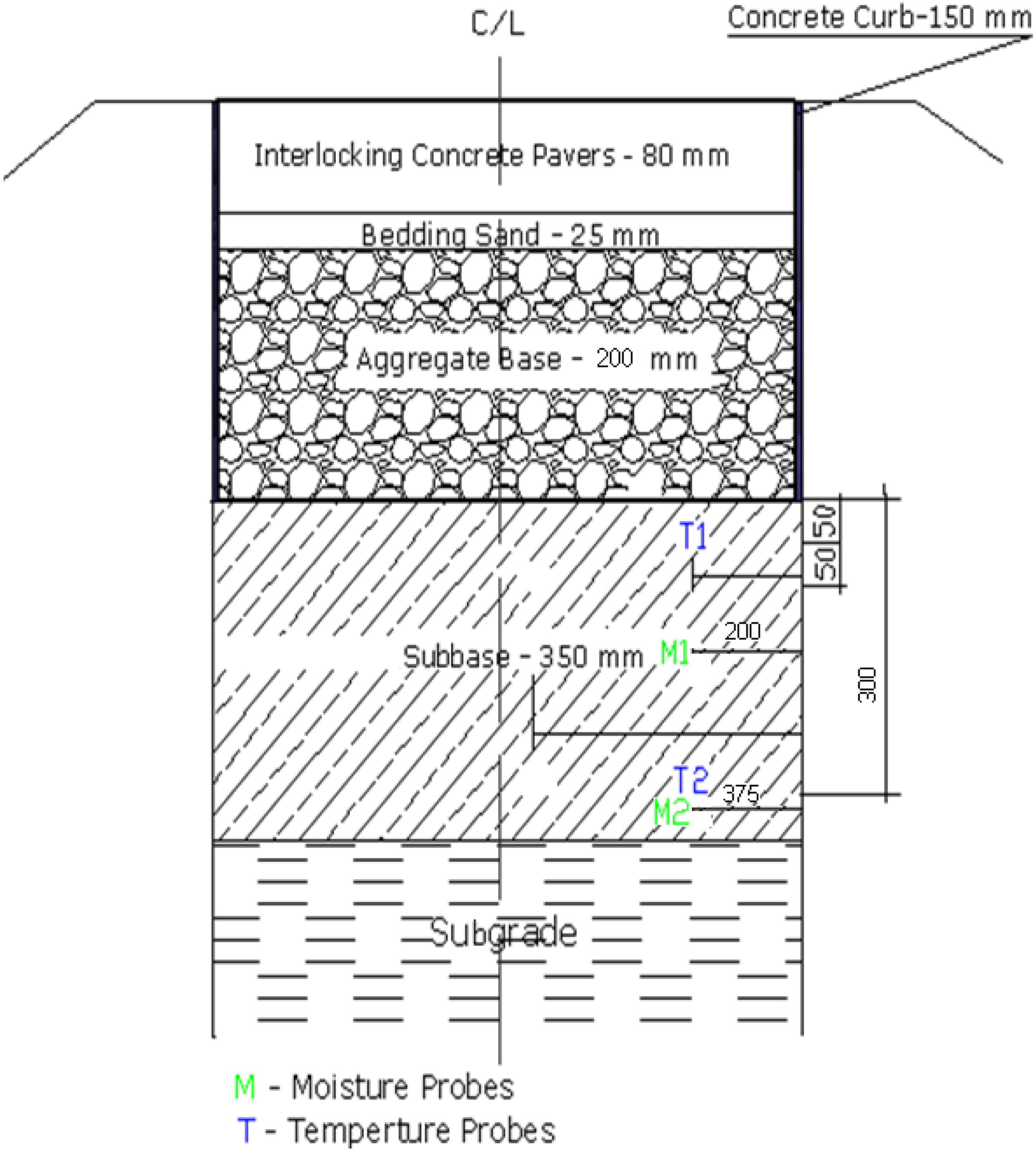


Figure 39 Transverse View of Instrumented Sand Set Granular Base Crosswalk (SSGBAH)

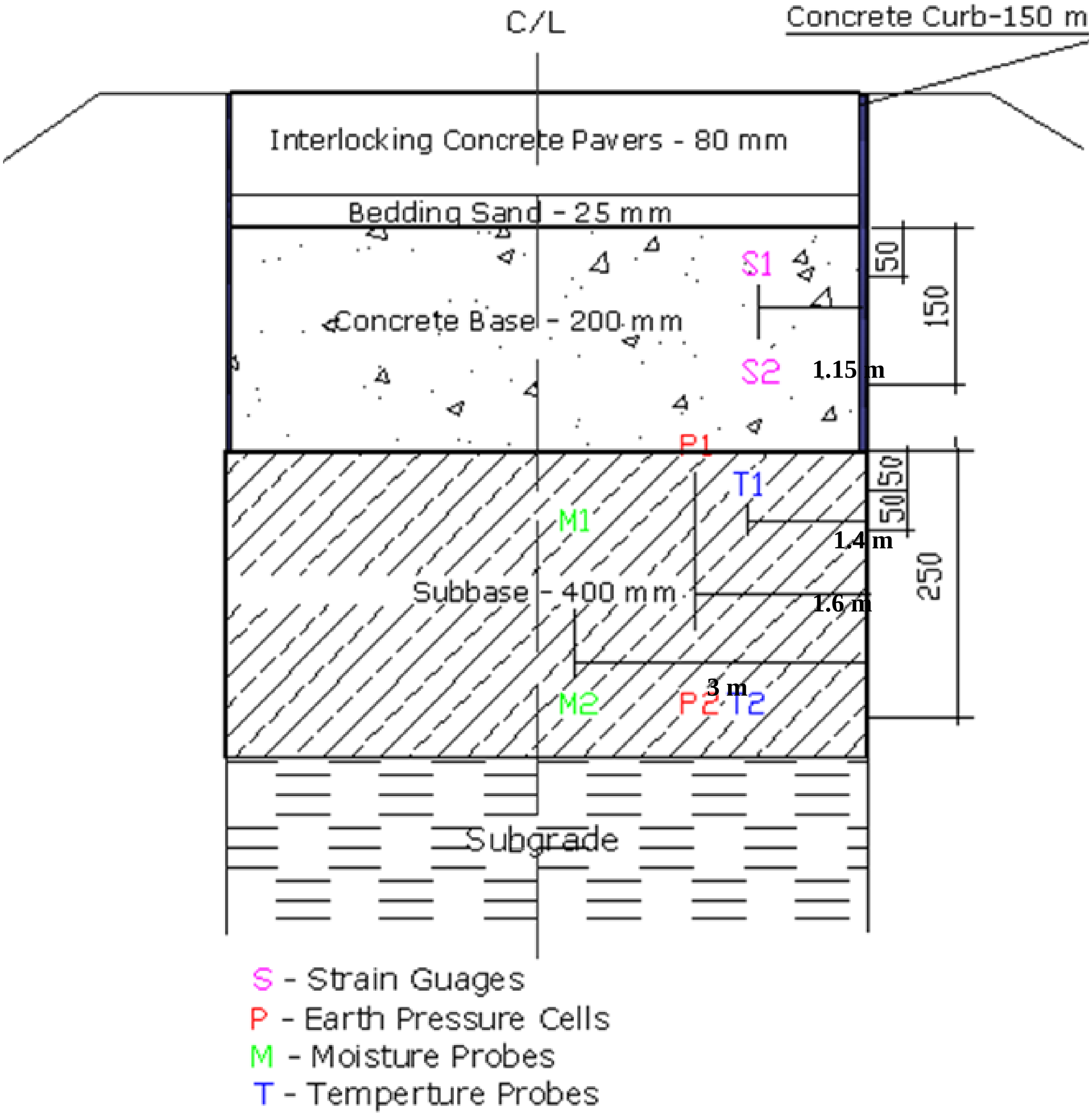


Figure 40 Transverse View of Instrumented Sand Set Concrete Base Crosswalk (SSCBCH)

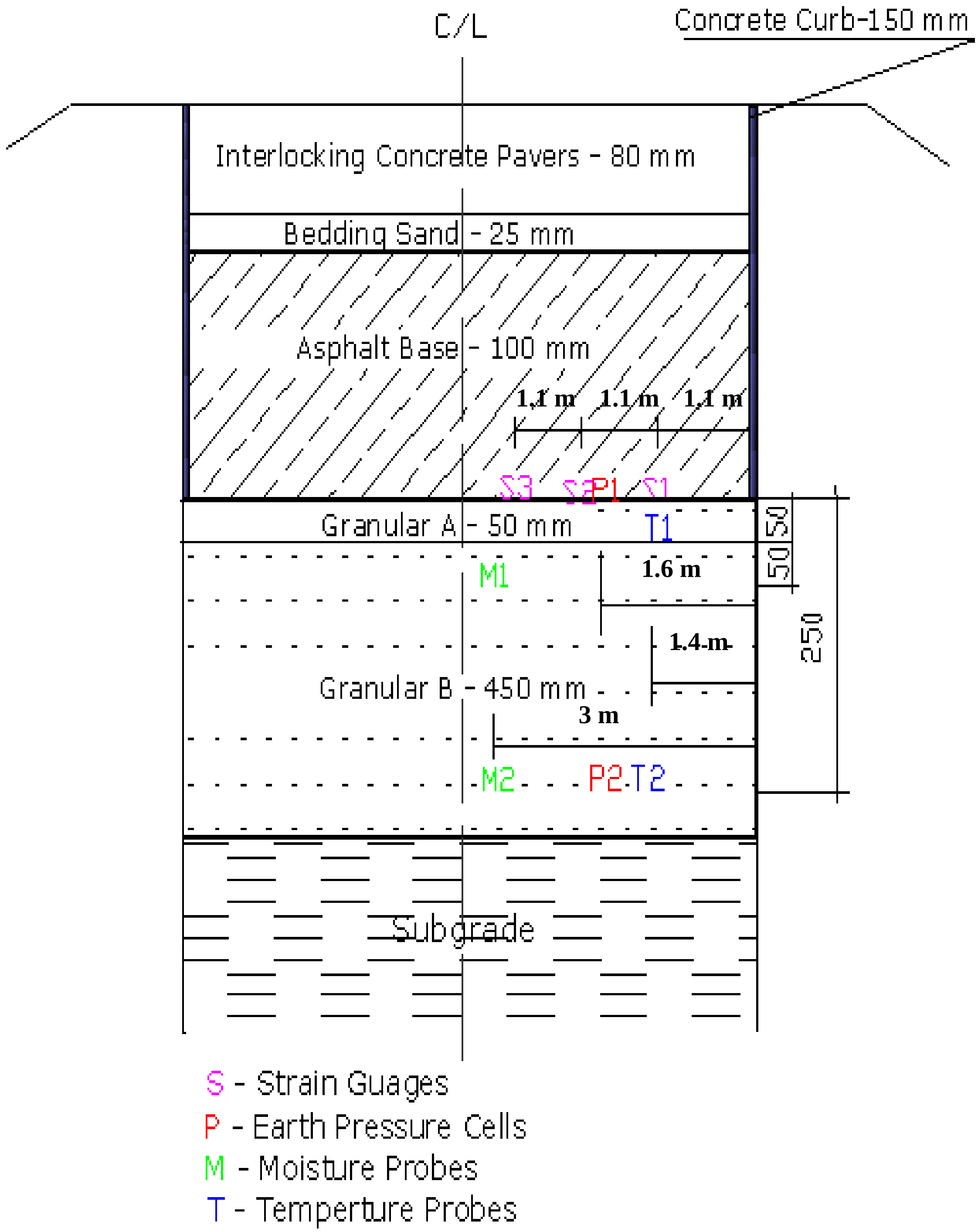


Figure 41 Profile View of Instrumented Sand Set Asphalt Base Crosswalk (SSABSH)

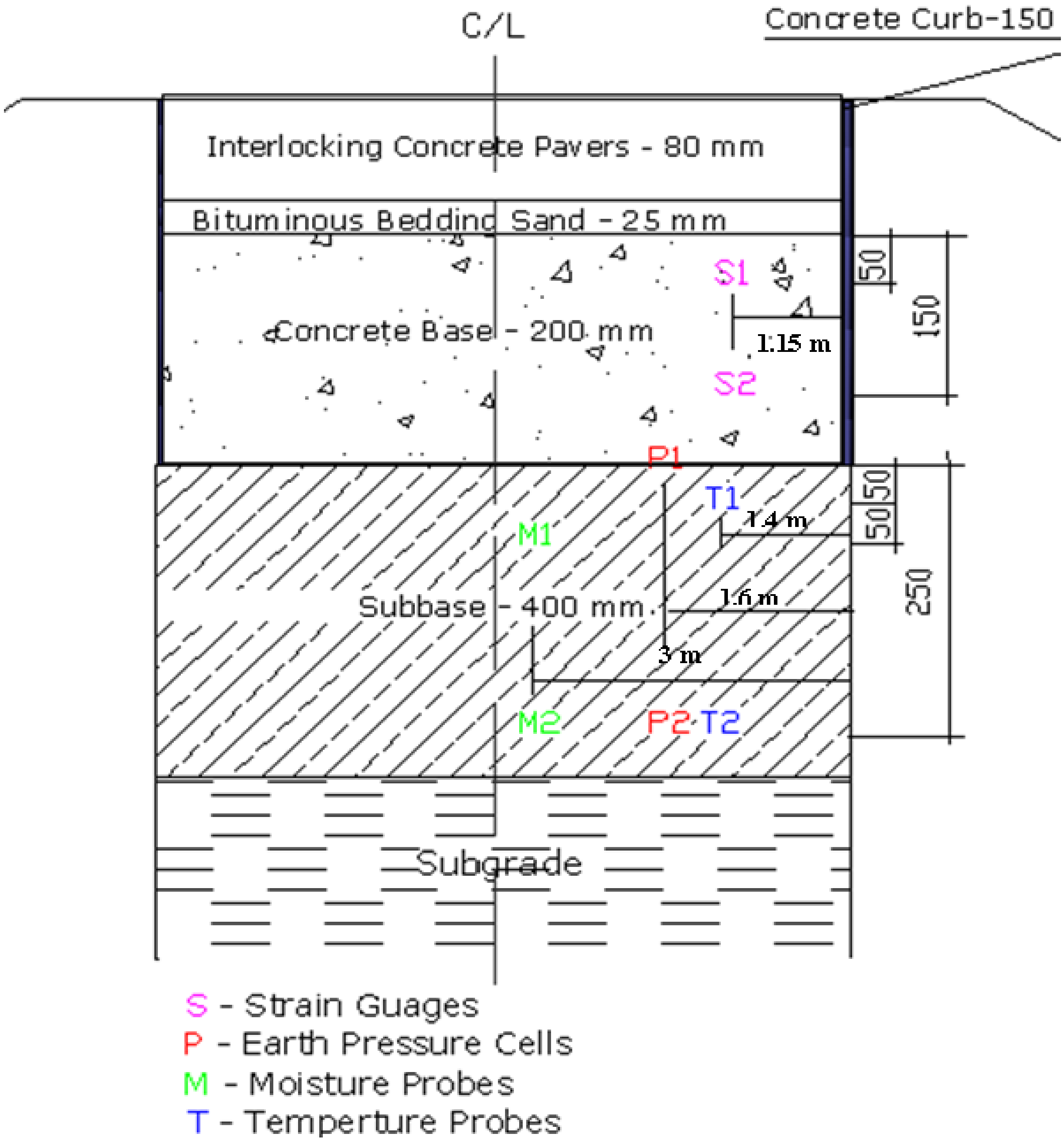


Figure 42 Transverse View of Instrumented Bituminous Set Concrete Base Crosswalk (BSCBCH)

3.1.6 Sampling and Testing

Extensive sampling was performed throughout the construction of the research project. Samples were taken from the original Granular A and Granular B and an HL3 sample was taken with the use of a shovel. Twelve concrete cylinders were casted to perform the laboratory compressive strength testing.

In addition to extracting samples, mix temperature of the HL3 and sand asphalt were also measured during the placement and before compaction. Slump and air-void testing were carried out for every batch of the concrete as shown in Figure 43.

Figure 43 On-site Concrete Air-Void Testing

Figure 44 shows the performance testing using the Portable Falling Weight Deflectometer (PFWD) that was undertaken on subbase, base and surface course layers on the project.

Figure 44 PFWD Testing on Crosswalk Subbase

3.1.7 Post Construction Phase

Laboratory compressive strength testing, laying conduits and feeding cables into conduits, connecting sensors with data logger are carried out in this phase.

Figure 45 A Cylinder after Compressive Strength Testing

3.1.7.1 Concrete Compressive Strength Test

During the concrete placement at the Test Track, twelve 100mm x 200mm concrete cylinders from each concrete mix were made. Two cylinders from each section were crushed using compressive strength testing machine at the 18 hrs, 24 hrs and on 2 days, 3 days, 7 days, 14 days, and 28 days after placement. The 28 day compressive strength for cylinders from both sections did not meet the 28 days compressive strength requirement (32 MPa) as shown in Figure 46.

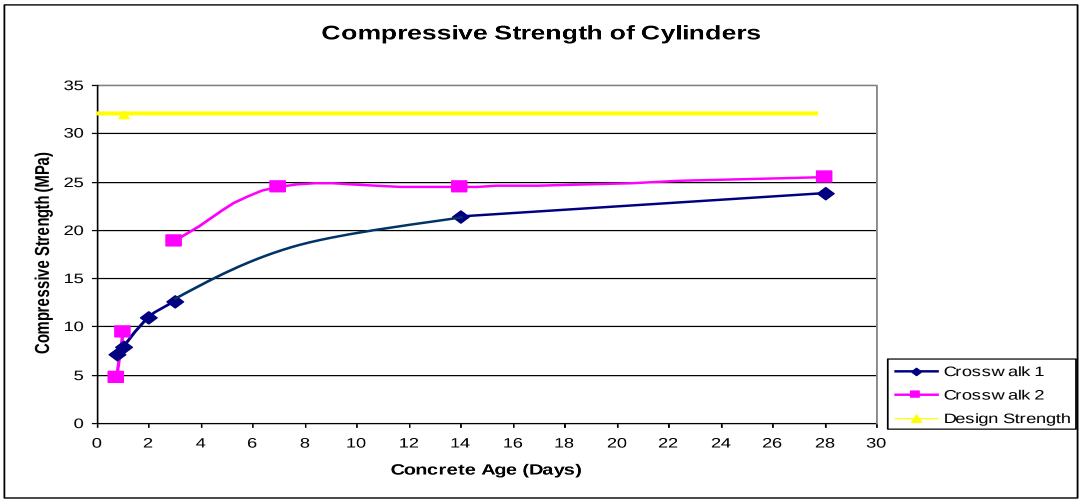


Figure 46 Compressive Strength Testing Results

3.1.7.2 Sensors Validation

After installation, all sensors were tested for functionality by connecting to the data logger. All of them were working normally during the testing. The data logger was installed on August 14, 2007 and all sensors have been connected to the data logger were working properly as shown in Figure 47.

Figure 47 A Preliminary Check of the Sensors after Installation

3.1.7.3 Conduit and Data Logger Installation

A 100mm diameter ABS conduit was used to route the cables to the data logger. A 50cm wide trench was excavated along the road embankment and the conduits were laid and buried with the excavated materials. The data logger is installed at 8.5m west of middle crosswalk section near the fence.

As for phase two, a separate conduit was run parallel to the phase one conduit and the sensors are connected to the same data logger as shown in Figure 48.

Figure 48 Conduit with Cables Running to Data Logger

3.2 University of Waterloo Ring Road

3.2.1 Project Location

The second series of crosswalks is situated at the University of Waterloo Ring Road as noted in Figure 49. There are four crosswalks with different bases and bedding materials located at UW north campus gate. The sand set crosswalk over concrete base is installed at station 2+010 Northbound, bituminous set over concrete base is at station 2+010 Southbound, the sand set over asphalt is at station 1+140 and the sand set over aggregate base is at station 1+095.

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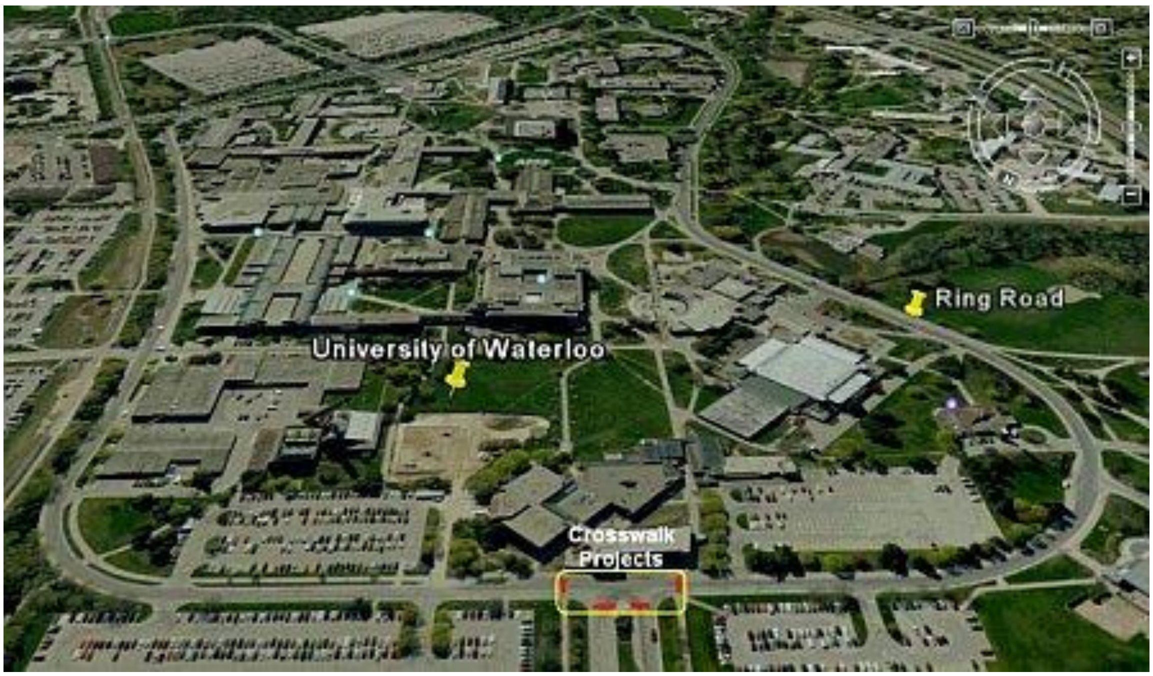


Figure 49 University of Waterloo Ring Road and Project Locations (Source: Google Earth)

The research projects were constructed during the resurfacing of the UW Ring Road. The length of asphalt base and aggregate base crosswalks are 12.5m and concrete base sections are 11.25m. The width of all sections is 2.635m. The construction was started on July 24, 2007 and completed on August 24, 2007. The installation of the interlocking concrete pavers was done manually by a crew of two to three members. Several sensors were installed to measure the response of the pavement in various locations during construction.

3.2.2 Reconstruction Phase

The preconstruction phase includes site selection, layout, sensors placement design and preliminary sensors testing. Three different types of sensors namely vibrating wire strain gauges, temperature profiles and moisture probes were designed to determine vertical strain in asphalt and concrete bases and temperature and moisture variation at different elevation in subbase.

A preliminary check of the sensors was made by connecting to the readout box and observing the displayed readout prior to the installation.

3.2.3 Pavement Section

The proposed crosswalk projects were constructed during the resurfacing of UW Ring Road. Two 50mm thick asphalt layers of HL3 and HL4 were used as surface and binder courses for the resurfacing. The structural components of the pavement are shown in Figure 50.

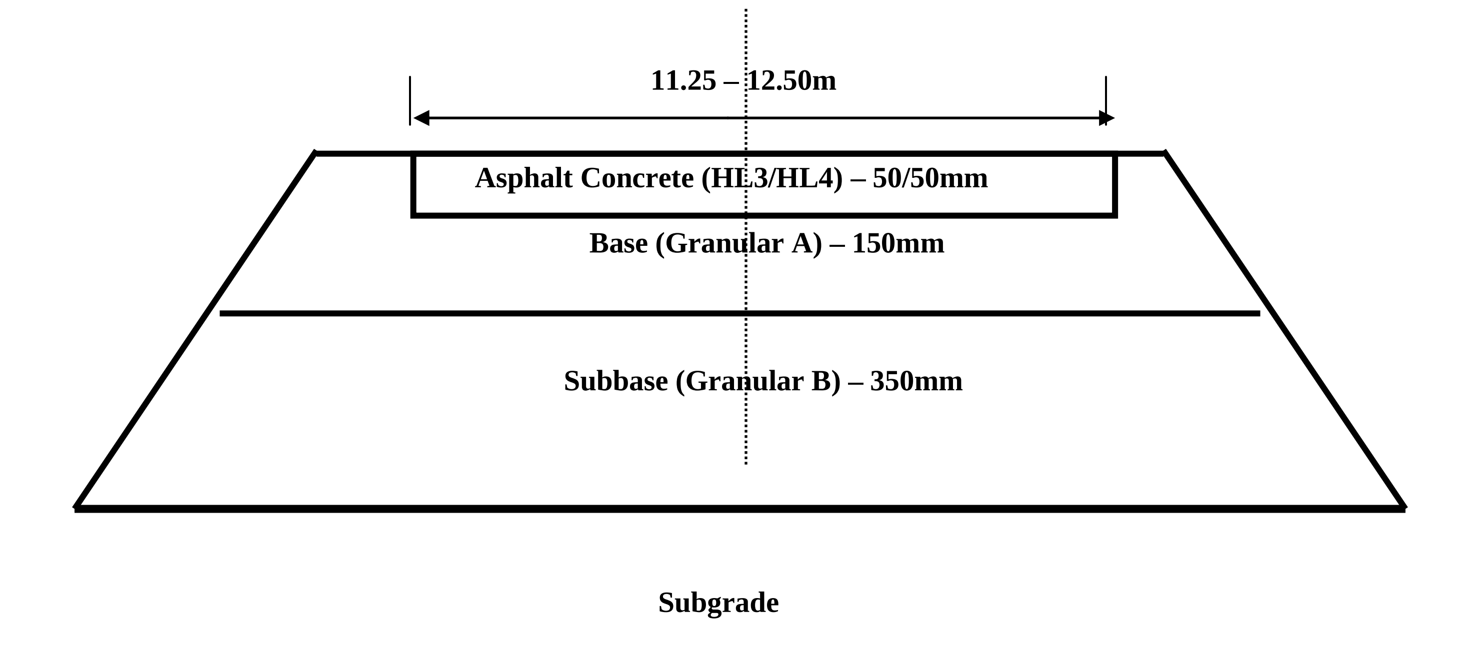


Figure 50 Existing Road Cross Section

A typical interlocking concrete paver’s crosswalk consists of concrete pavers placed on top of a layer of bedding sand over a base and subbase layers. Four different designs selected for this study are as follows and are shown in Figure 51.

- Sand Set pavers over Concrete Base with Concrete Header (SSCBCH)
- Bituminous Set pavers over Concrete Base with Concrete Header (BSCBCH)
- Sand Set pavers with Aluminum Header over Asphalt Base (SSABAH)
- Sand Set pavers with Concrete Header over Granular Base (SSGBCH)

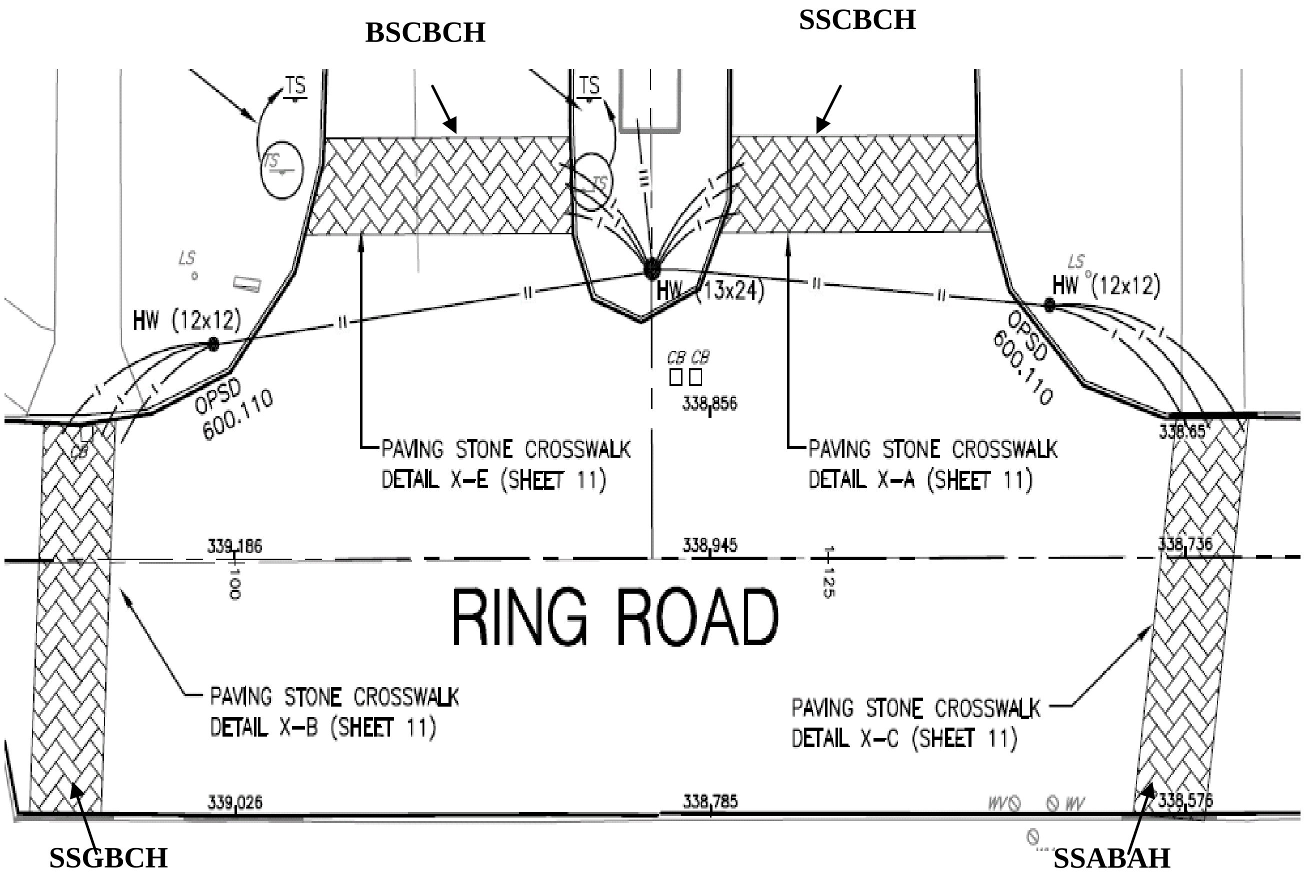
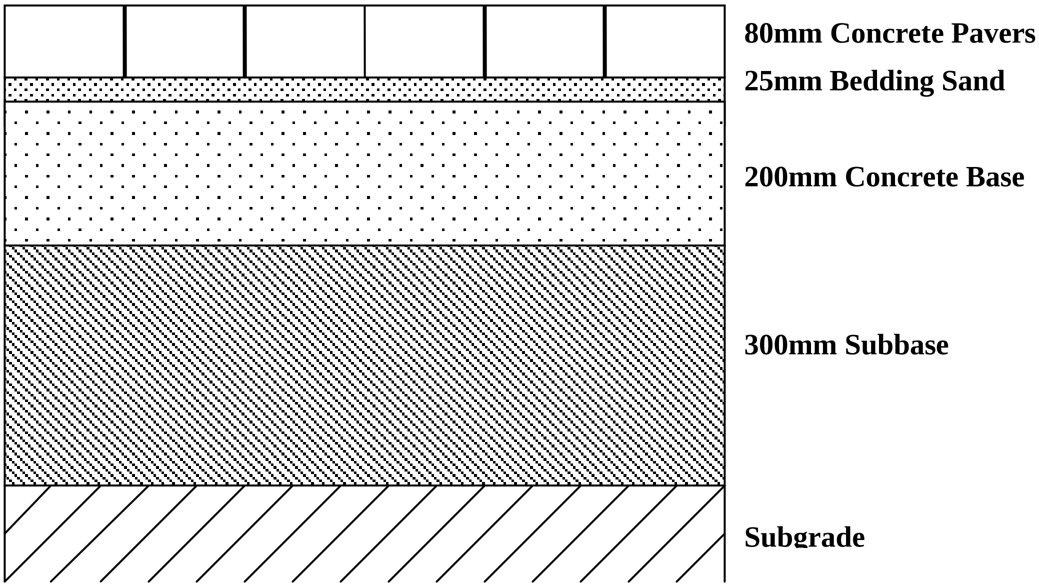


Figure 51 Layout of Crosswalk Test Sections at UW Ring Road



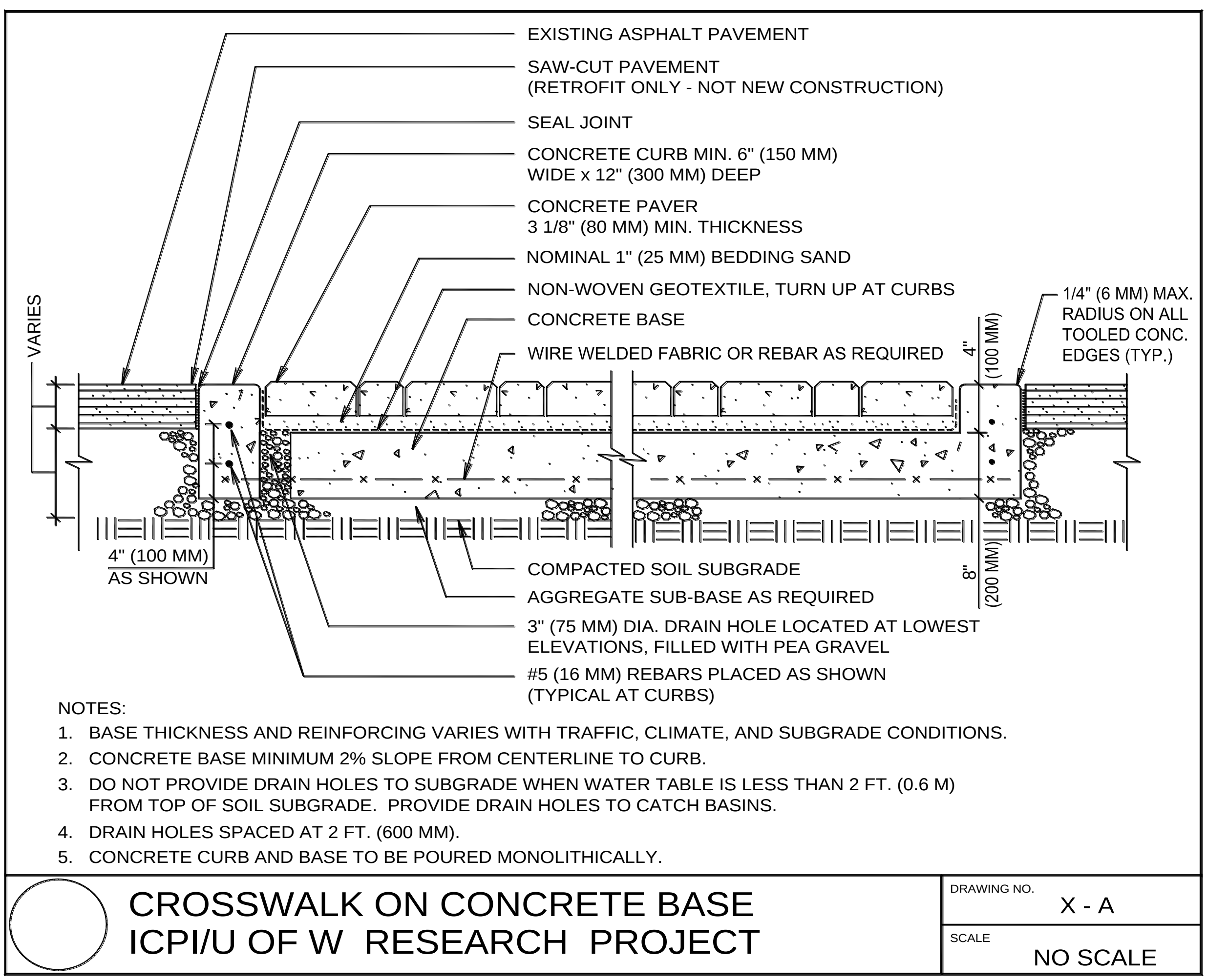
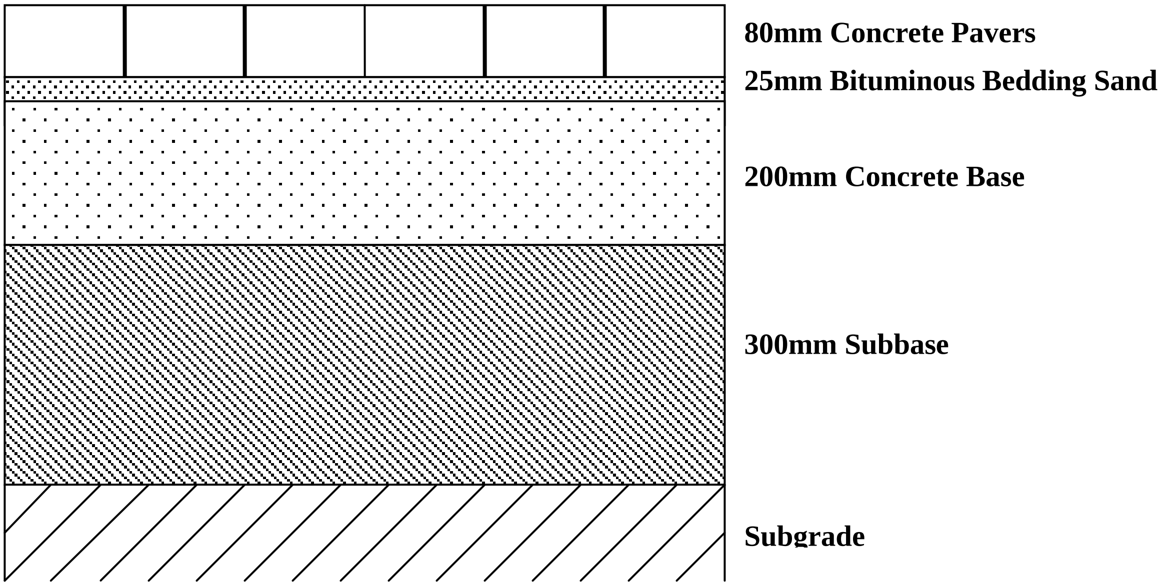


Figure 52: Sand Set Concrete Base Crosswalk Section (SSCBCH)



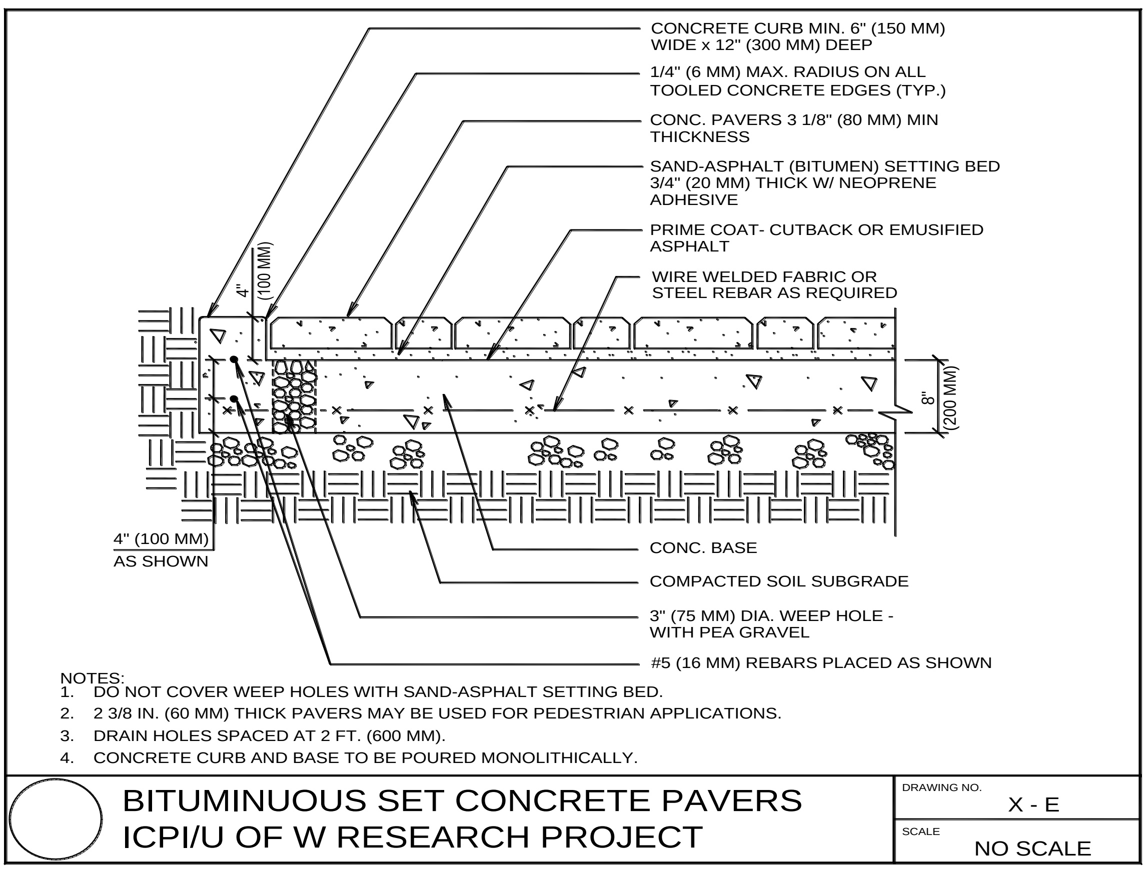
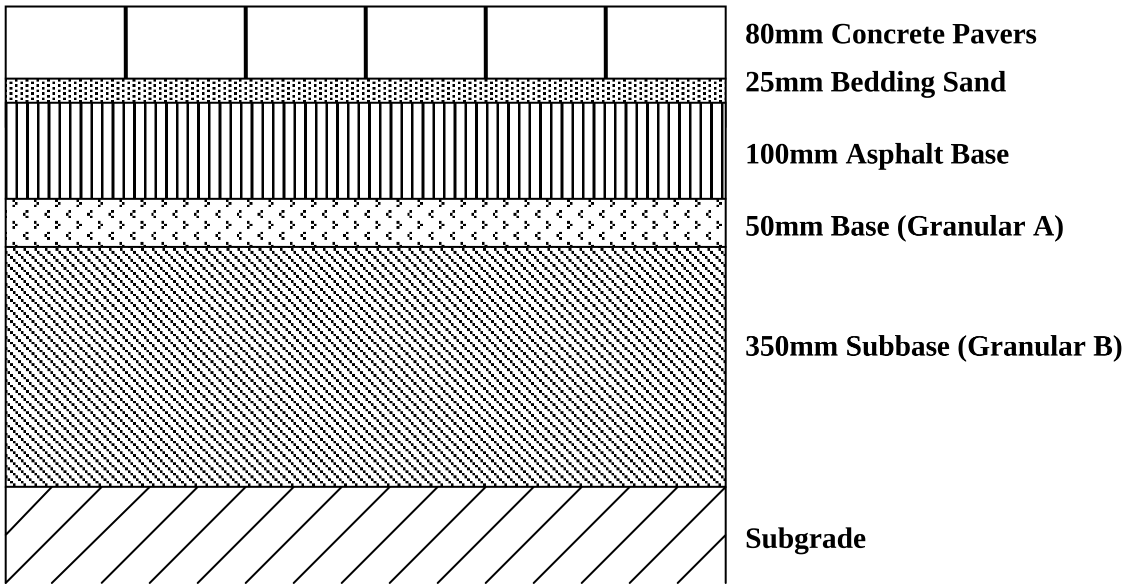


Figure 53 Bituminous Set Concrete Base Crosswalk Section (BSCBCH)



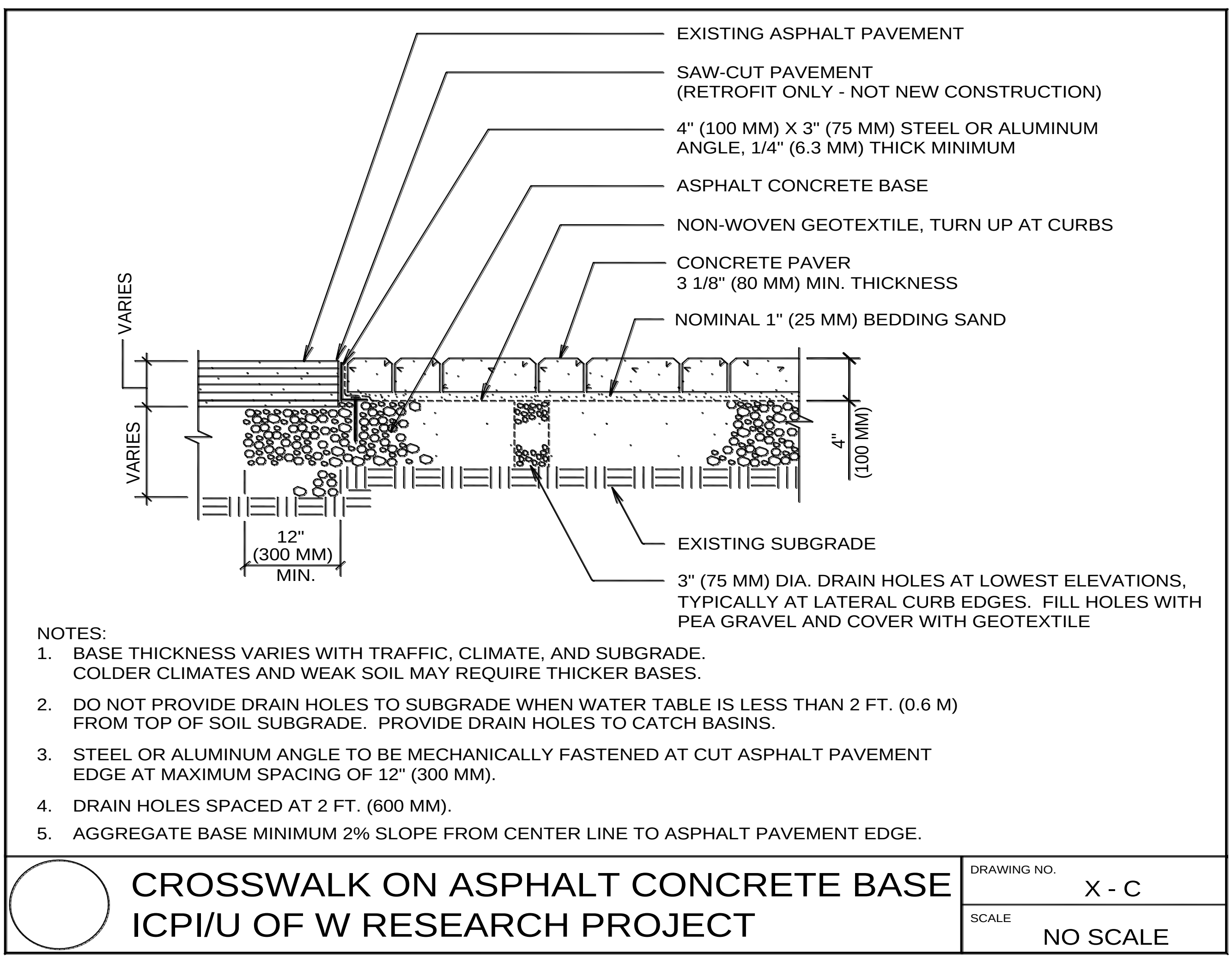
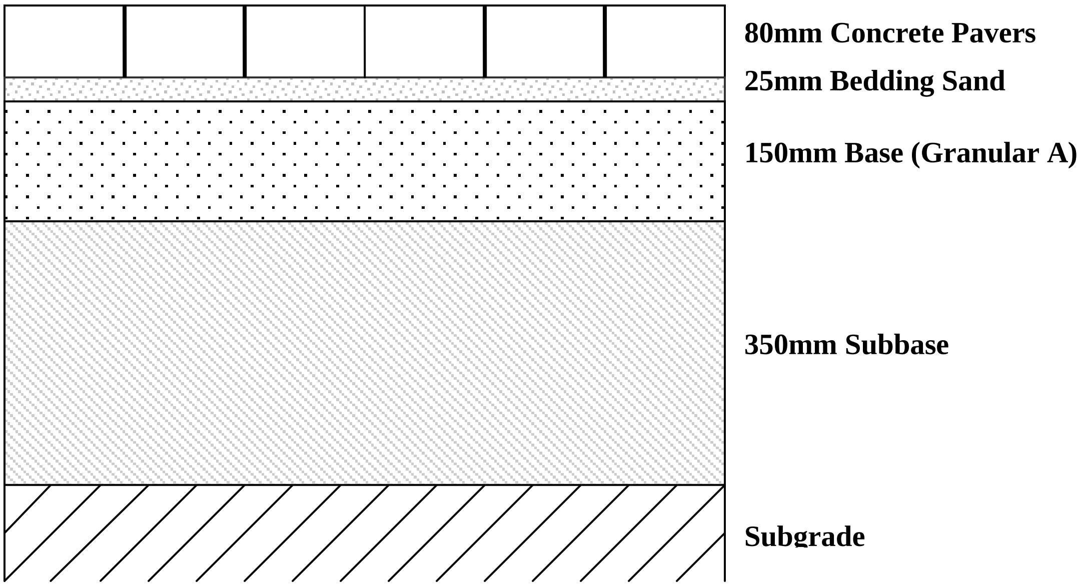


Figure 54 Sand Set Asphalt Base Crosswalk Section (SSABAH)



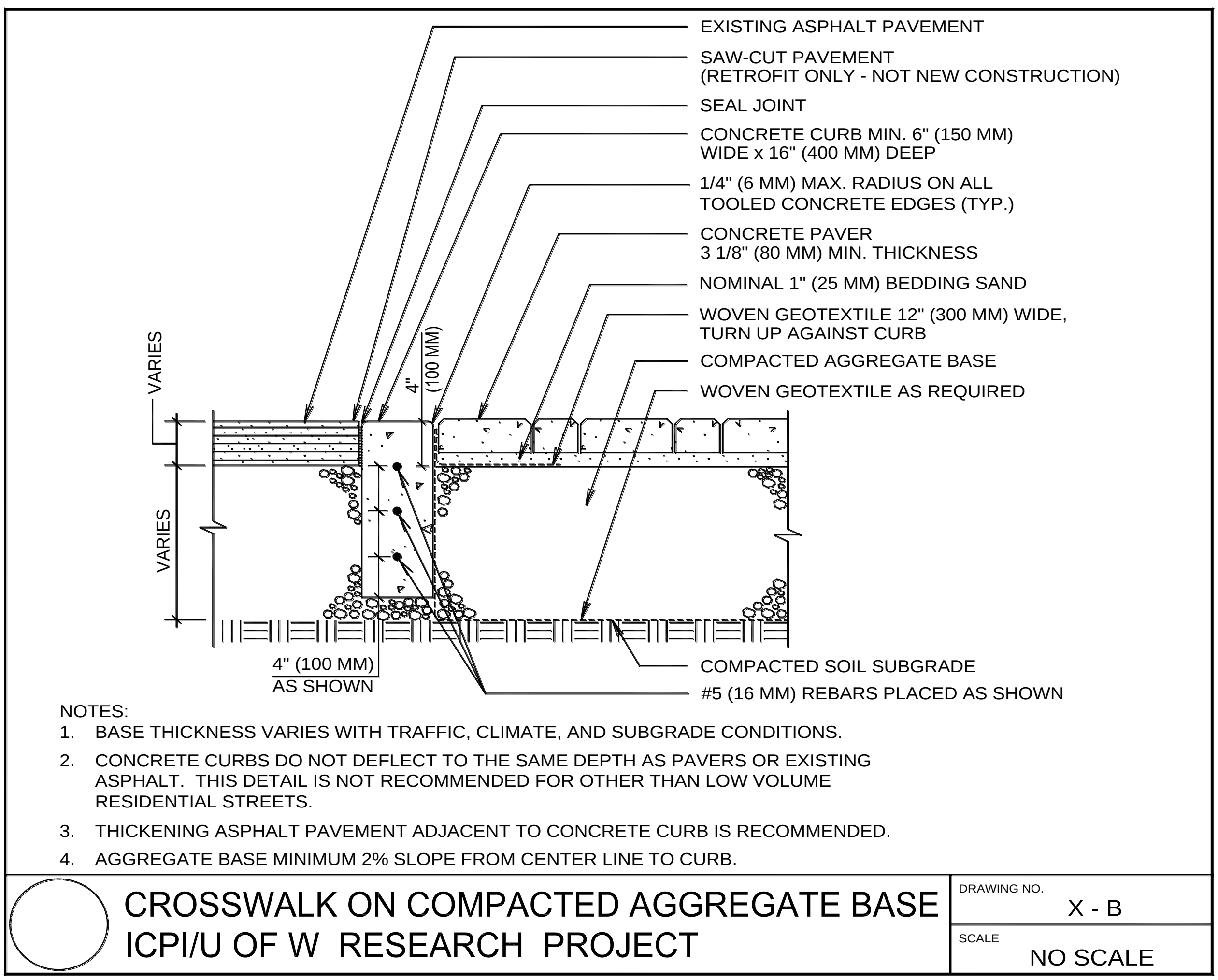


Figure 55 Sand Set Granular Base Crosswalk Section (SSGBCH)

3.2.4 Construction Phase

The construction of the CPATT interlocking concrete crosswalks at the UW Ring Road were carried out by Ross Yantzi's Pavestone Plus Limited and the interlocking concrete pavestone were supplied by NAVA STONE in Cambridge, Ontario. The entire construction phase lasted approximately one month. Each of the crosswalks are 2.635m in width and concrete base crosswalks are 11.25m and asphalt base and aggregate base are 12.50m in length. The sand set crosswalk over concrete base was installed at station 2+010 Northbound, bituminous set over concrete base is at station 2+010 Southbound, the sand set over asphalt is at station 1+140 and the sand set over aggregate base is at station 1+095. Table 6 below summarizes the construction activities.

Table 6 Summary of Construction

Crosswalk	Date	Construction Activity
SSCBCH	July 23, 2007	Excavation of subbase for concrete base, moisture and temperature probes installation
	July 23, 2007	Compaction of subbase course and formworks for curbs and headers
	July 24, 29, 2007	Reinforcement rebars and drainage pipes placement
	July 24, 2007	Strain gauges installation and concrete placement
	July 27, 2007	Non-woven geotextile installation and spreading and screeding of bedding sand
	July 27-31, 2007	Installation of ICP
	July 31, 2007	Spreading and sweeping of joint sands and final compaction
BSCBCH	July 25, 2007	Excavation of subgrade for concrete base, moisture probes and temperature probes installation
	July 25, 2007	Compaction of subbase course and formworks for curbs and headers
	July 25, 2007	Reinforcement rebars and drainage pipes placement
	July 26, 2007	Strain gauges installation and concrete placement
	August 1, 2007	Non-woven geotextile installation and spreading and screeding of bituminous sand along with coating of asphalt-neoprene adhesive
	August 1, 2007	Placement of ICP
	August 1, 2007	Spreading and sweeping of joint sands and final compaction
SSABAH	July 24, 2007	Excavation of subbase and moisture probes and temperature probes installation
	July 24, 2007	Compaction of subbase course
	July 25, 2007	Installation of strain gauges, asphalt (HL4) paving
	August 24, 2007	Installation of Aluminum angle edge restraint
	August 24, 2007	Non-woven geotextile installation and spreading and screeding of bedding sand
	August 24, 2007	Installation of ICP
	August 24, 2007	Spreading and sweeping of joint sands and final compaction
SSGBCH	July 26, 2007	Excavation of subbase and installation of moisture and temperature probes
	July 27, 2007	Compaction of subbase course and formworks for curbs and headers
	July 28, 2007	Concrete placement for curbs and headers
	June 31, 2007	Non-woven geotextile installation and spreading and screeding of bedding sand
	June 31, 2007	Installation of ICP
	June 31, 2007	Spreading and sweeping of joint sands and final compaction

3.2.4.1 Excavation of the Subbase for Base Construction and Sensors Installation

The first phase in the Ring Road crosswalk construction involved excavating the subbase layer for the resurfacing of the road as shown in Figure 56. After this was completed, the first crosswalk section used a backhoe excavator to excavate the newly built Granular B subbase layer. The cutting of the section started at approximately 8:00 am on July 23, 2007 and lasted for two hours. Immediately following the cutting, the moisture and temperature probes were installed in the designed locations in the subbase and backfilled and compacted with a small plate compactor.

Excavation of BSCBCH, three and four was carried out on July 24, 25 and 26, 2007 respectively. Backfill of the trench and compaction were done immediately following the installation of moisture and temperature probes.

Figure 56 Excavation of Subbase

3.2.4.2 Crosswalk Base Construction

Within the four crosswalk test sections, SSCBCH and two have a concrete base underneath the ICP while SSABAH has an asphalt base and SSGBCH has an granular base underneath the ICP. The concrete base crosswalk (SSCBCH) was constructed first. After the final compaction of the subbase with a 20 kN/ 4500 lbs vibrating plate compactor, wooden forms were installed around the perimeter of the crosswalk for curbs and headers. Reinforcement rebars having mesh of 150mm /150mm were placed on the top of the subbase course before the concrete placement as shown in Figure 57. High Early Concrete from Cross Country Concrete Ontario Limited, Heidelberg was brought to the construction site at 10:10 AM on July 24, 2007 and placed into the forms from 10:16 am to 10:50 am. A total of 7.0m^3 of concrete poured in place over half an hour period.

Figure 57 Formworks, Drain Pipes and Reinforcement Bars for Concrete Base Section

The concrete base of BSCBCH was constructed on July 26, 2007. High Early Concrete from Cross Country Concrete Ontario Limited, Heidelberg was brought to the construction site at 7:15 AM and poured into the forms from 7:20 AM to 7:55 AM. A total of 7.0m^3 of concrete poured in place over thirty five minutes period as shown in Figure 58.

Figure 58 Concrete Placement

The length of the concrete base sections is 11.25m and the width is 2.635m including the headers on both sides is shown in Figure 59. The thickness of the concrete bases is 200mm. Each base has curb and headers around the perimeter, the width of the curb/header is 150mm. Exposed concrete headers edges are trimmed to 3mm radius to reduce the likelihood of chipping. The curing agent was applied after the initial setting of the concrete.

Prior to the concrete placement, twenty four ABS pipes with diameter of 75mm and length of 300mm were installed perpendiculars to the road surface at the lowest level of the base course. Since the road base is constructed with a two percent slope towards the edge, four pipes were installed along the two outer edges of the bases and sixteen pipes were placed longitudinally along the inner side of the crosswalk. The pipes were filled with pea gravel and covered with non-woven geotextile to prevent the loss of bedding materials. Their purpose is to remove excess water from the bedding course. The distance between adjacent drainage pipes (centre to centre) is approximately 60cm and the first drainage pipe of each set is 20cm offset from the edge of the base.

Figure 59 Concrete Base Section

The construction of the asphalt base of SSABAH was started on July 25, 2007 at 10.10 am and took approximately one hour to complete the first 50mm lift. Within the one hour of paving, 8.0 tonnes of HL4 was spread, screeded, and compacted with a 20 kN plate compactor. The paving of the second/final 50 mm lift of HL4 was started on the same day at 11:00 am and completed at 12:30 pm. The air temperature during this operation ranged from 28°C to 33°C. The mix temperature on arrival to the site was 148°C and 126°C before compaction of the first lift. Similarly, the mix temperature of the second lift before the compaction was 109°C. The mix was prepared at Cambridge Asphalt Supply of Steed and Evans Limited. Construction is shown in Figure 60.

The asphalt base also has two concrete curbs similar to the concrete bases along the two edges that are parallel to the Ring Road. Two sets of four ABS pipes were placed along the outer edges of the base and filled with pea gravel and covered with geotextile. The distance between adjacent drainage pipes (centre to centre) is approximately 60cm and the first drainage pipe of each set is 20cm offset from the edge of the base.

Figure 60 Asphalt Base Construction

The construction of the aggregate base SSGBCH was started on July 26 2007, and the installation of moisture and temperature probes in subbase, 150mm thick Granular A base course was placed and compacted with a 20 kN plate compactor. The concrete curbs and headers were built on July 27, 2007 as shown in Figure 61.

Figure 61 Aggregate Base Construction

3.2.4.3 Bedding Material Placement

Prior to placing the interlocking concrete pavestone, a layer of bedding material was placed in the base of each section. Gradation of the bedding sand has already been provided in Table 1. For crosswalks with concrete base (SSCBCH) and granular base (SSGBCH), a layer of non-woven geotextile was placed in the constructed base and a layer of bedding sand with approximate thickness of 25mm was spread and screeded on top of the non-woven geotextile.

For asphalt base crosswalk (SSABAH), L-shaped aluminum angles were installed on top of the asphalt base and along the cut pavement edge. The aluminum angle is 9.5cm wide and 9.5cm high and 6 mm thick. Nails and screws were drilled alternatively through the aluminum angle and into the asphalt base 60cm apart to fasten the edge restraint onto the base. On top of the angle plate and the asphalt base, a similar layer of non-woven geotextile and bedding sand were placed.

For crosswalk BSCBCH, the concrete base surface was prepared with an emulsified asphalt tack coat. A hot-sand asphalt mix was brought to the site and spread and compacted to 25mm thick layers. After the asphalt cooled, a thin coating of asphalt-neoprene adhesive was applied across the surface.

Figure 62 Bedding Sand and Non-woven Geotextile

3.2.4.4 Placement of Interlocking Concrete Pavers (ICP)

The interlocking concrete pavers were placed into the SSCBCH, two, three and four on July 27, 2007, August 1, 2007, August 24, 2007 and July 31, 2007 respectively. Interlocking concrete pavers from NAVA STONE in Cambridge, Ontario with approximate dimension of 200mm × 100mm × 80mm were placed in 45° herringbone pattern. Installation was carried out in an identical manner to the test sections at the CPATT Test Track started from the corner with securing 45° string lines on the bedding course. Edge pavers are saw cut to fit against the sailor and soldier courses. The sailor and soldier are interlocking concrete pavers which were placed in such a way that each section has three sides (one long side and two short sides) with sailor course and one long side with soldier course. After the installation, the surface was compacted with a vibratory plate roller to compact the bedding sand, seat the pavers in it and force the bedding sand into the joints at the bottom of the pavers. The procedure is presented in Figure 63. The final elevation of the surface course was kept 6mm above the adjacent asphalt pavement to accommodate any future settlement.

Figure 63 Laying Interlocking Concrete Pavers

3.2.4.5 Joint Sands Placement and Compaction

After the initial compaction of the pavers was completed, dry joint sand was spread on the surface and compacted with a vibratory compactor to ensure that the spaces between pavers are filled. Excess joint sand was later removed.

Figure 64 Concrete Header and Joint Sand

3.2.5 Instrumentation

The sensors were installed during the construction of the project**.** Three different types of sensors namely vibrating wire strain gauges, temperature profiles and moisture probes were designed for concrete and asphalt base sections. In contrast, only two types of sensors namely moisture and temperature probes were used in the aggregate base section. The sensors were installed at different locations inside the pavement and all cables were collected at one point and fed into conduits and routed to the proposed data logger station. Table 7 summarizes the purpose and locations of the sensors.

Table 7 Summary of Instrumentation

Crosswalk	Sensor	Quantity	Location	Purpose
1 (SSCBCH)	Concrete Strain Gauge (VWSGE)	2	50mm and 150mm below top of concrete base at 1.15m offset from road edge	Measures change of strain in concrete
	Thermistor (THO003-250-2)	2	50mm and 250mm below bottom of concrete base at 1.4m offset from road edge	Measures change of temperature
	Moisture Probes (LPTPCO9-V)	2	100mm and 250mm below top of concrete base at 3m offset from road edge.	Measures in-situ moisture content
2 (BSCBCH)	Concrete Strain Gauge (VWSGE)	2	50mm and 150mm below top of concrete base at 1.15m offset from road edge	Measures change of strain in the concrete
	Thermistor (THO003-250-2)	2	50mm and 250mm below bottom of concrete base at 1.4m offset from road edge	Measures change of temperature
	Moisture Probes	2	100mm and 250mm below top of concrete base at	Measures in-situ

	(LPTPCO9-V)		3m offset from road edge.	moisture content
3 (SSABAH)	Asphalt Strain Gauge (VWSGEA)	3	Bottom of the asphalt base at 1.1m, 2.2m and 3.3m offset from road edge	Measures change of strain in the asphalt
	Thermistor (THO003-250-2)	2	50mm and 250mm below bottom of asphalt base at 1.4m offset from road edge	Measures change of temperature
	Moisture Probes (LPTPCO9-V)	2	100mm and 250mm below bottom of asphalt base at 3m offset from road edge	Measures in-situ moisture content
4 (SSGBCH)	Thermistor (THO003-250-2)	2	50mm and 250mm from bottom of concrete base at 1.4m offset from road edge	Measures change of temperature
	Moisture Probes (LPTPCO9-V)	2	100mm and 250mm below the bottom of the granular base at 3m offset from road edge	Measures in-situ moisture content

3.2.5.1 Moisture Probes

A moisture probe is a device consisting of three 600mm long stainless steel rods for measuring moisture content as shown in Figure 65. Two probes are installed horizontally in each section at 100mm and at 250mm from the top of the subbase and at 3m offset from the road edge.

To install a moisture probe, a cavity of 700mm x 200mm x 300mm is excavated into the subbase to accommodate the moisture probe. A cable trench of 200mm wide x 250mm deep running from the cavity to the edge of pavement was also excavated as shown previously. This trench is used to run the cables of pressure cells and temperature probes as well. All sharp stone fragments were removed from the cavity and the trench. The cavity and the trench were filled with 5mm sand layer and the probes and cables were placed. After placing the sensors and cables the trench was filled with sand and subbase materials and compacted with a Marshall hammer to ensure the density. Similarly, the second probe is installed at 150mm above the first probe and filled with sand and compacted with a Marshall hammer. The cables were run along the trench together and then fed into conduit.

Figure 65: Installation of Moisture Probes

3.2.5.2 Temperature Probe

Temperature probes are installed to measure temperature variation at different elevations within the pavement structure. The temperature probe consists of two thermistors which are 200mm long and are installed vertically in the subbase in each section. The thermistors are located at 50mm and 250mm from the top of the subbase at 1.4m offset from the road edge. Figure 66 shows the top of the probe after installation.

To install the temperature probes, a cavity of 300mm x 300mm x 600mm is excavated into the subbase. All sharp stone fragments were removed from the cavity and the trench. The cavity and the trench were filled with 5mm sand layer and the probes and cables were placed. After placing the temperature probe and the cables, the trench was filled with sand and subbase materials and compacted with a Marshall hammer to ensure the density. The cable was routed along the trench and then fed into conduit.

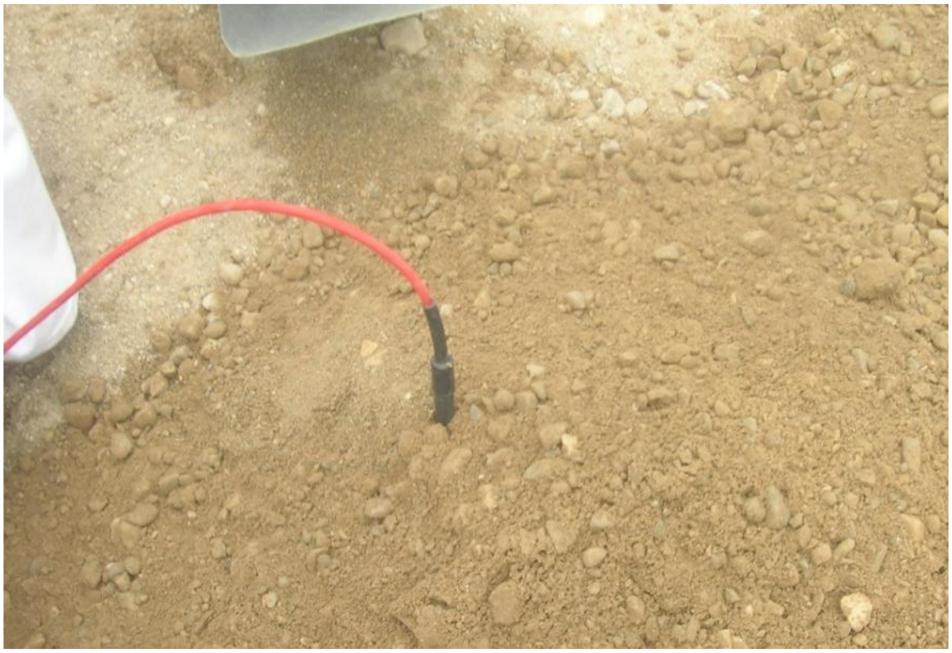


Figure 66 Temperature Probe Installation

3.2.5.3 Vibrating Wire Strain Gauge

Two types of strain gauges were installed to measure a strain in asphalt and concrete bases as shown in Figures 67 and 68 respectively. Three asphalt strain gauges were placed at the bottom of the asphalt base at 1.1m, 2.2m and 3.3m offsets from the road edge. Alternatively, concrete strain gauges were installed at the depth of 50mm and 150mm from the top of the concrete base at 1.15m offset from the road edge.

A HMA pad was placed at the asphalt strain gauges locations. After the asphalt cooled, the strain gauges were hand placed and gently pressed into the mix as shown in Figure 67. A shallow cable trench was excavated and routed the cables along the trench and fed into conduit. The trench then was filled with sand and compacted with the Marshall hammer.

Figure 67 Asphalt Strain Gauge Installation

Figure 68 Vibrating Wire Concrete Strain Gauges Installation

Concrete strain gauges were attached with U-shaped chairs and the chairs were driven into the ground and tied together to prevent from movement as shown in Figure 68. A shallow cable trench was excavated to run the cable and finally fed into conduit. The trench was then filled with sand and compacted with the Marshall hammer.

Profile views including installation are provided in Figures 69, 70, 71 and 72.

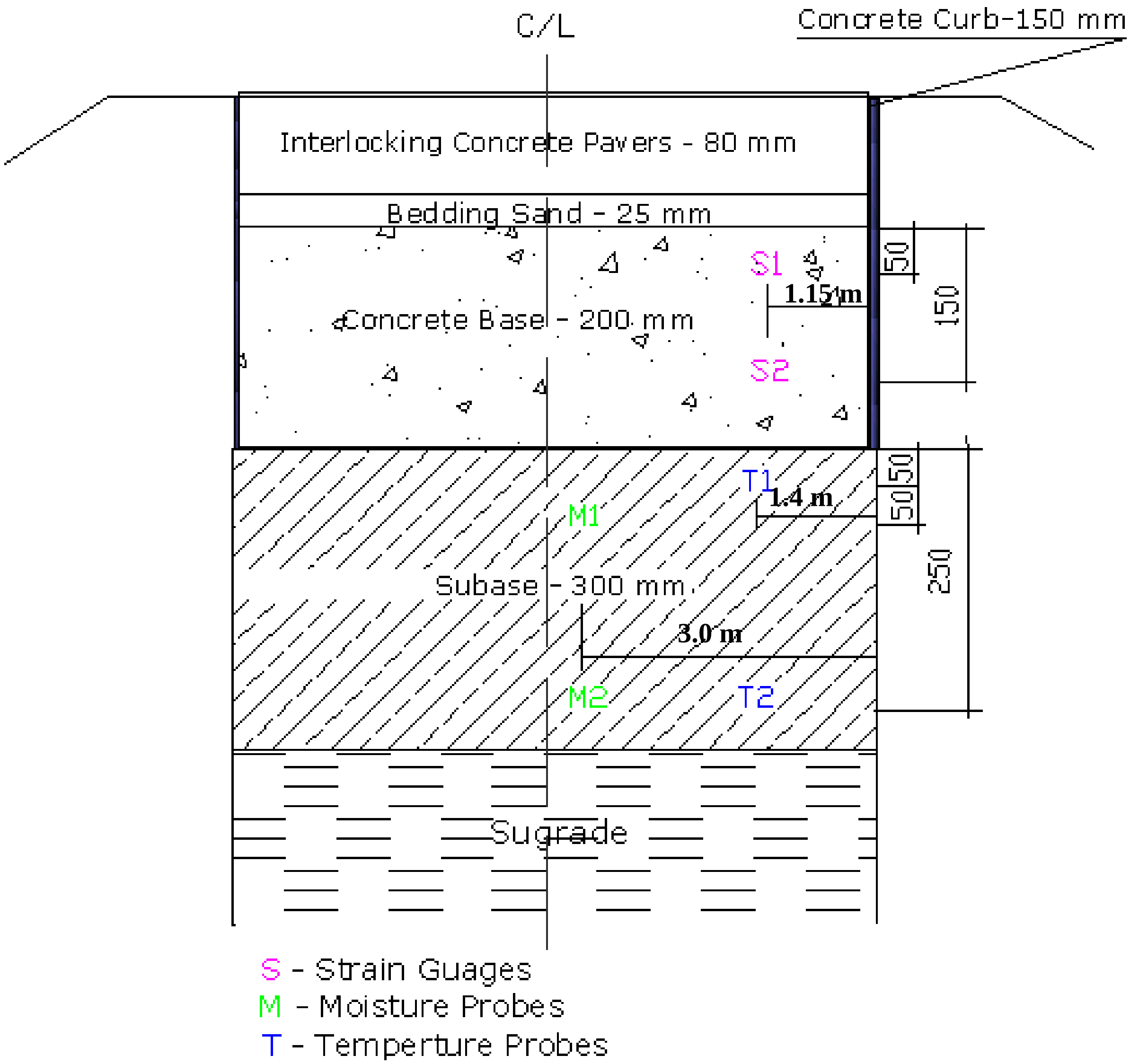


Figure 69 Transverse View of Instrumented Sand Set Concrete Base Crosswalk (SSCBCH)

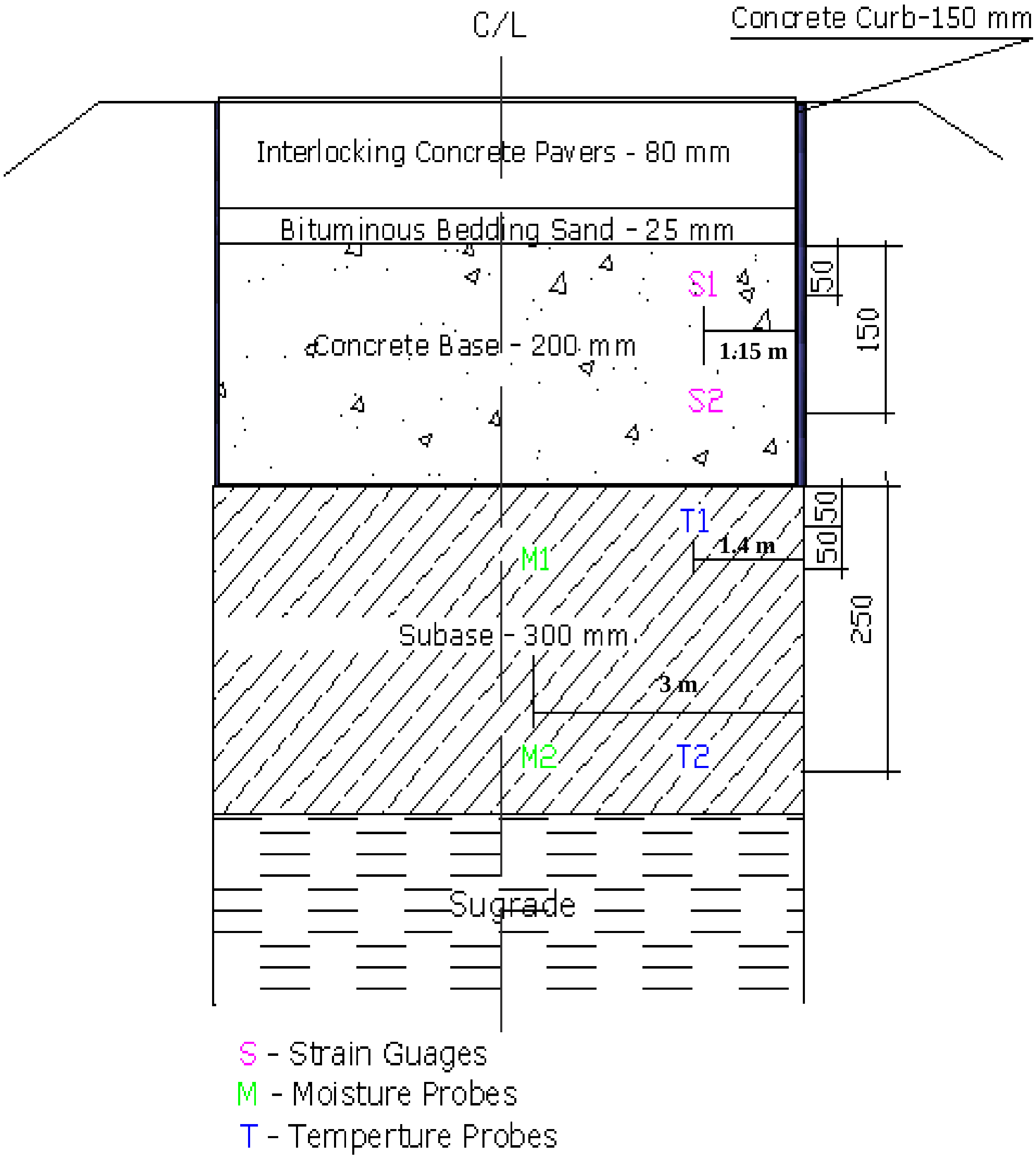


Figure 70 Transverse View of Instrumented Bituminous Set Concrete Base Crosswalk (BSCBCH)

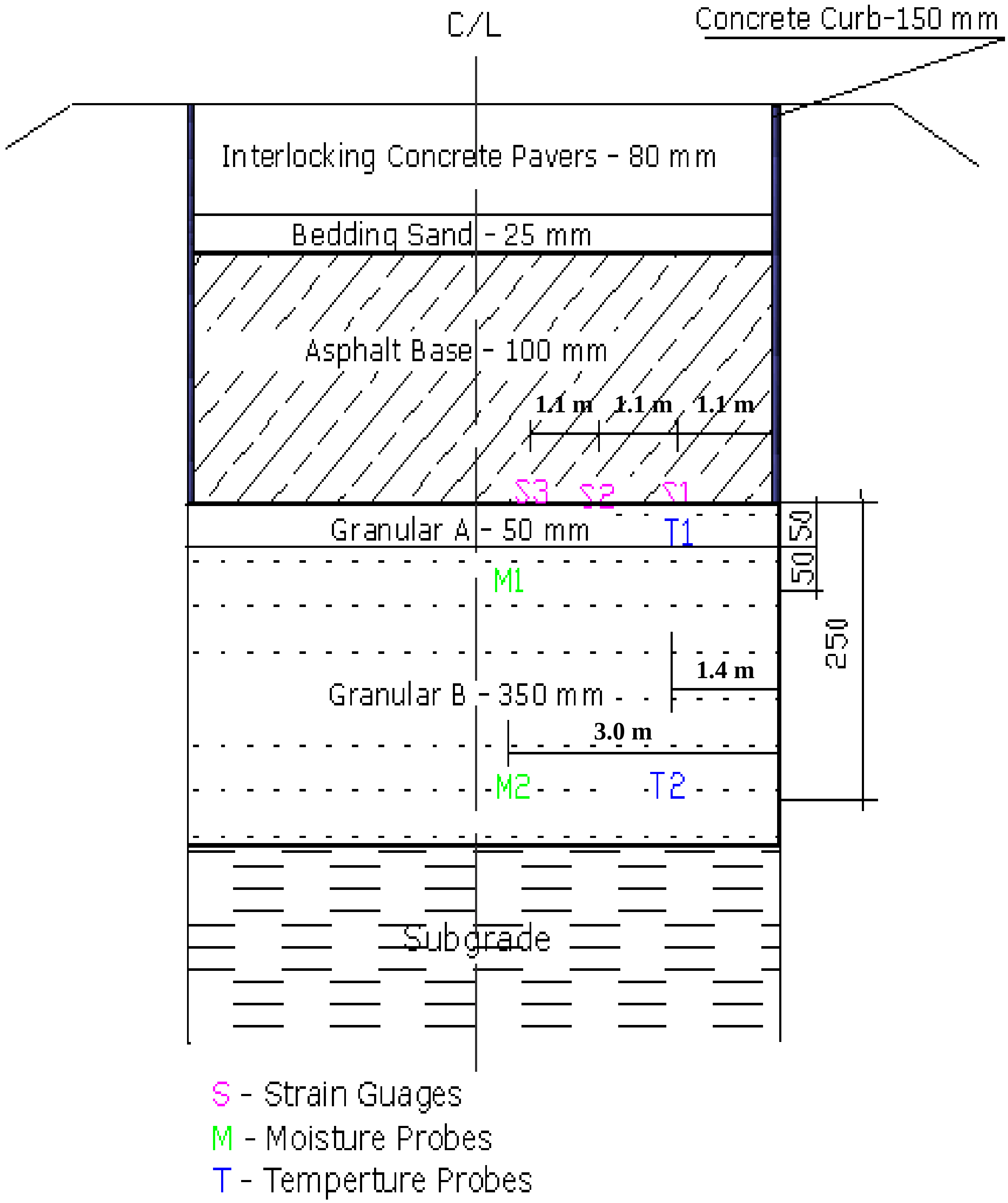


Figure 71 Transverse View of Instrumented Sand Set Asphalt Base Crosswalk (SSABAH)

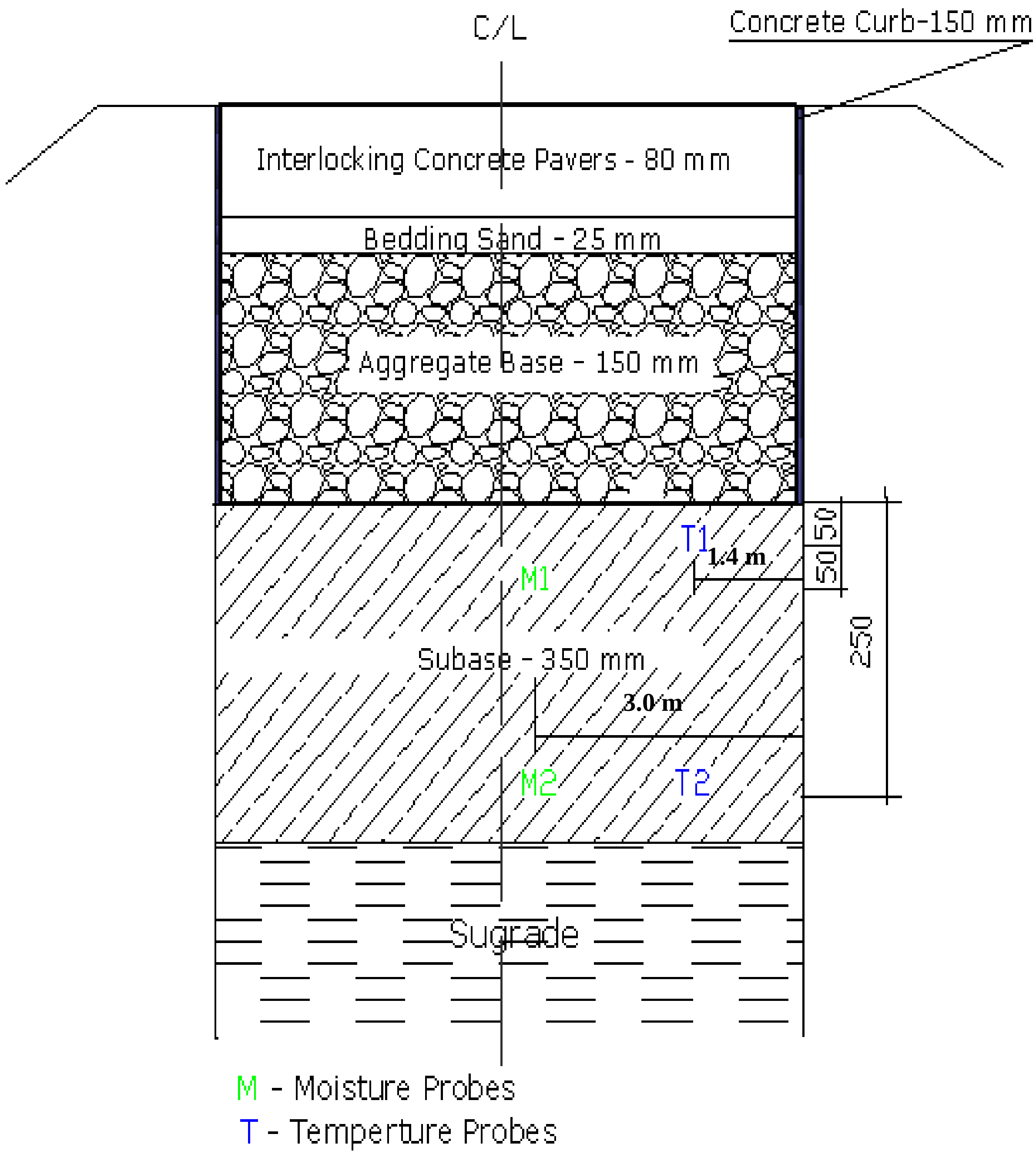


Figure 72 Transverse View of Instrumented Sand Set Granular Base Crosswalk (SSGBCH)

3.2.6 Sampling and Testing

Extensive sampling was performed throughout the construction of the research project. Samples were taken from the original Granular A and Granular B and an HL4 sample was taken with the use of a shovel. Fifteen concrete cylinders were casted to perform the laboratory Compressive Strength Testing.

In addition to extracting samples, mix temperature of the HL4 and sand asphalt were also measured during the placement and before compaction. Slump (Figure 73) and air-void testing were carried out for every batch of the concrete.

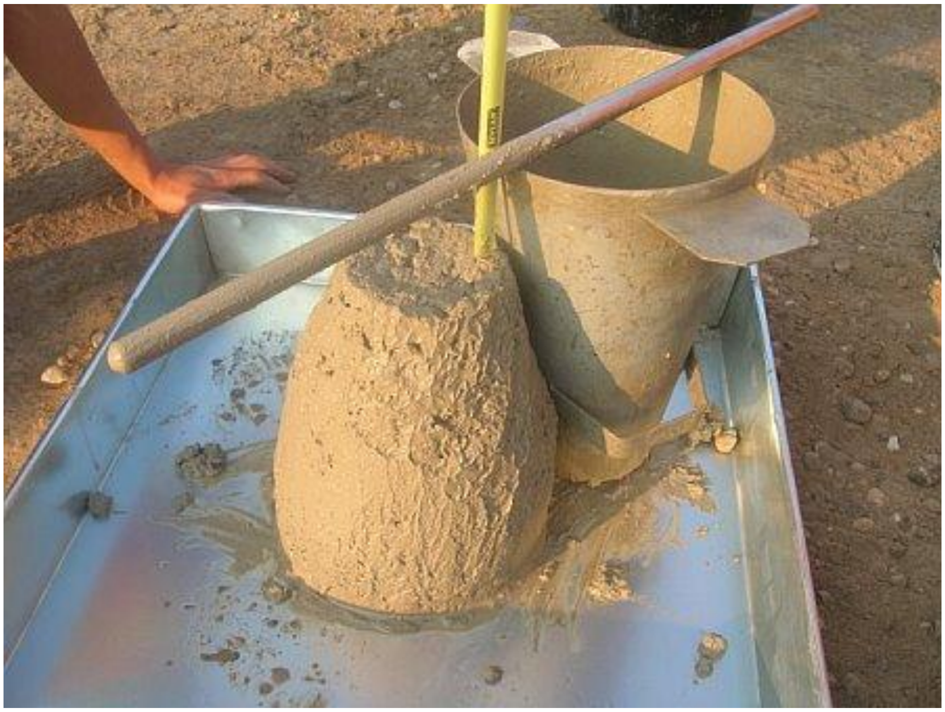


Figure 73 On-site Concrete Slump Testing

Performance testing using Portable Falling Weight Deflectometer (PFWD) was carried out on all test sections.

3.2.7 Post Construction Phase

Laboratory compressive strength testing, Figure 74, laying conduits and feeding cables into conduits were carried out in this phase.

3.2.7.1 Concrete Compressive Strength Test

During the concrete placement at the Test Track, twelve 100mm x 200mm concrete cylinders from each concrete mix were made. Two cylinders from each section were crushed using compressive strength testing machine on 1 days, 2 days, 3 days, 7 days, 14 days, and 28 days after placement. As shown in Figure 75, the 28 day compressive strength for cylinders from both sections did meet the 28 days compressive strength requirement (32 MPa).

Figure 74 A Cylinder After Compressive Strength Testing

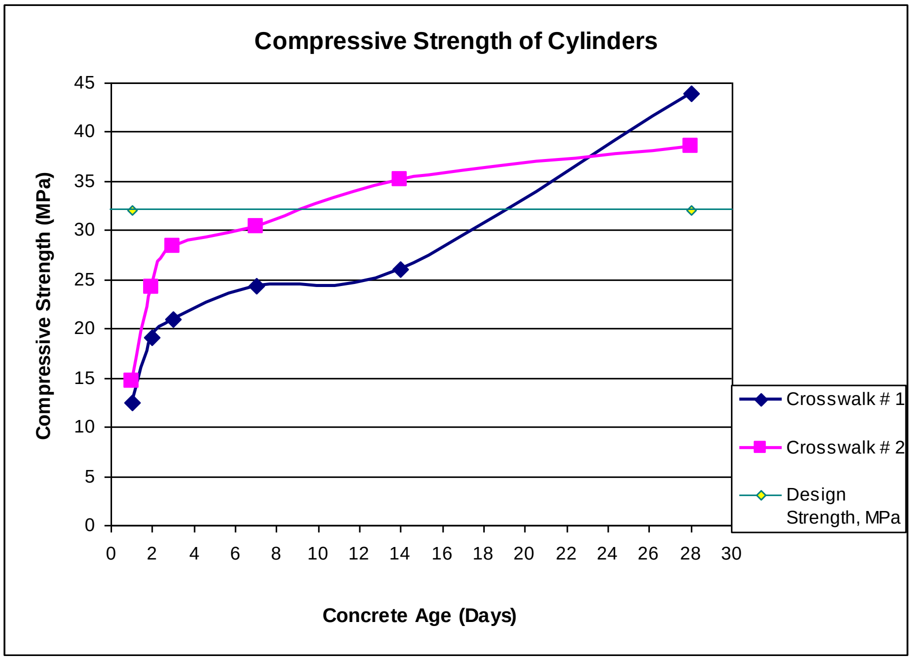


Figure 75 Compressive Strength Testing Results

3.2.7.2 Conduit Layout and Data logger Installation

The ABS conduits were 75mm in diameter and placed to route the cables from each crosswalk section to the collection point as shown in Figure 76 and a 100mm diameter conduit was installed to route all cables to the data logger from the collection point. The data logger was installed in between SSCBCH and

BSCBCH on the road island. The data logger box is housed in a traffic cabinet which is placed on an existing concrete sidewalk and fastened with bolts Figure 76 shows the plan view of conduits and data logger layout.

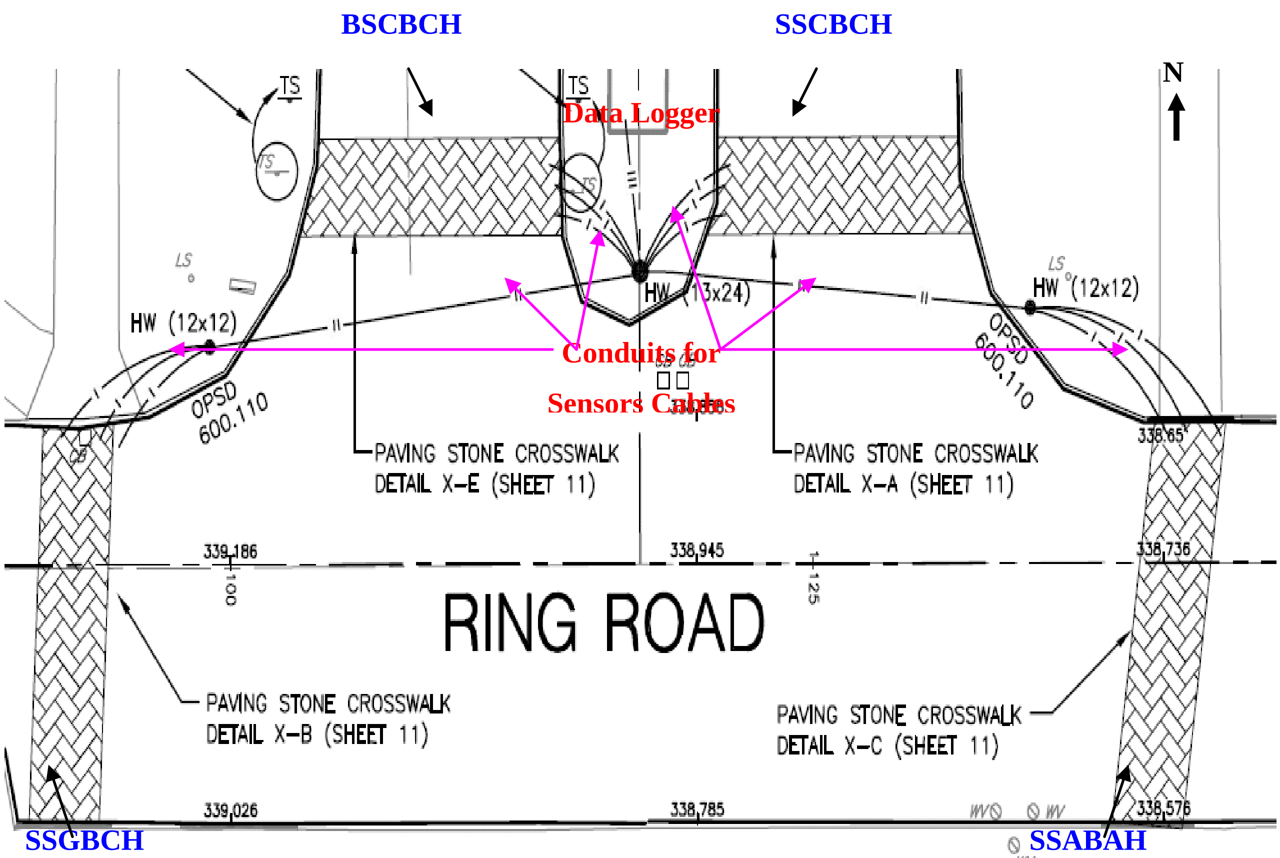


Figure 76 Conduits and Data Logger Layout

Chapter 4

Crosswalk Repair / Maintenance

4.1 Repair of Sand Set Asphalt Base Steel Header at Test Track

4.1.1 Traffic Control

Construction of the Sand Set Granular Base Aluminum Header (SSGBAH) crosswalk and repair to the existing crosswalk Sand Set Asphalt Base Steel Header (SSABSH) was carried out during a four-day period. There was continuous flow of traffic at the Test Track, so, traffic control consisting of two flag persons was required and other students at CPATT assisted with this. Construction was carried out one lane at a time. To ensure safety of workers on the field, traffic control operation was performed for the four full days. Figure 77 shows the traffic control operation with a flag person

Figure 77 Traffic Control Operations

4.1.2 Survey

In a survey done in July, 2009 out of the three crosswalks at the Test Track the second one which is Sand Set Asphalt Base Steel Header failed under existing loading conditions and repair was necessary. This crosswalk was constructed on July 2007. A conference call was held on July 5, 2009 between the CPATT research team, Ross Yantzi the contractor and the ICPI technical committee to discuss about the condition and possible repair method. After the discussion, it was decided to remove the pavers and examine possible failure causes. As per discussion, Susan Tighe, Ross Yantzi and other CPATT members the visited Test Track to investigate the cause of failure. Figure 78 shows the crosswalk's picture before pavers were lifted.

Figure 78 Crosswalk Before Lifting the Pavers

Figure 79 Crosswalk After Lifting the Pavers

The pavers with the soldier course which were against the steel edge were lifted and it was found that no bedding sand was remaining under the pavers. The geotextile in that area had also failed. Figure 79 shows the section right after the paver was lifted. However, the area under the paver, bedding sand and geotextile were intact and were performing well. It can be seen clearly in Figure 79, the sand is in good condition. It had shoved, but the gradation was very good. After a discussion, the team concluded that the reason for failure of this crosswalk at the edges is due separation of the steel edges from the asphalt edge which would vibrate when vehicles went over. As a result of the vibration, the steel edges moved and eventually resulted in the movement of pavers.

Consequently, the joint sand also was lost allowing for the infiltration of water into the pavers, which resulted in bedding sand being pumped out. The sample of bedding sand from the failing section and the good section were collected and gradation test was performed. The summaries of test results are provided in Table 8. All except three of the sieves are within the percent passing. The slightly higher percentage passing of the failed sample on the No. 100 sieve is most probably due to contamination.

Table 8 ASTM C33 Requirements and Actual Values

Sieve Size	Percent Passing	Good Performing Area	Failed Area
3/8 inch	100	98.1	97.4
No. 4	95-100	93.4	93.6
No. 8	85-100	75.8	76.6
No. 16	50-85	60.1	61.5
No. 30	25-60	40.9	45.4
No. 50	10-30	18.6	25.5
No. 100	2-10	6.0	13.1
No. 200	0-1	2.2	6.7

A field report was presented by Ross Yantzi to the team and was discussed on the conference call on July 17, 2009. The decision was made to install an aluminum header, new geotextile, bedding sand and pavers.

The ESALs on the crosswalk for the period of 25 months starting from construction to repair would be 3.125 million. This crosswalk showed a sign of failure during May 2009. The rain of 2009 and the vehicle flow caused the crosswalk to deteriorate quickly.

4.1.3 Repair Work

As described in the earlier paragraph, the header was changed from steel to aluminum. The maintenance work was done along with the construction of the new crosswalk. The same methods and materials were used as for the SSGBAH. Following the repair this test section was renamed to reflect the change in the header. Please refer to section Construction of SSGBAH for the detail description of each work. However there are a few variations which are summarized in Table 9.

Table 9 Summary of Repair Work to SSABSH

Crosswalk	Section	Date	Construction Activity
SSABAH	Southbound (loaded lane)	July 22,2009	Removal of Pavers
			Marked 610mm from the edge of the SSABSH crosswalk
			Removing of bedding sand, geotextile and steel header in SSABSH
			Installation of Aluminum header
			Non-woven geotextile installation and spreading and screeding of bedding sand
			Installation of ICP
			Compaction of the pavers
			Spreading and sweeping of joint sand stabilizer and final compaction
		July 23, 2009	Saw cut of asphalt on both sides
		July 24,2009	Placement of Asphalt patch on both sides of the crosswalk in southbound lane

Table 9 summarizes the construction work in the southbound lane. The construction of northbound lane is not explained here because the process was similar.

In the southbound lane during maintenance, the good pavers were cleaned and reused while any bad or failed pavers were replaced with new pavers. The joint sand stabilizer was used instead of joint sand. The asphalt was cracked on the edges of the crosswalk. In order to ensure a better load transfer, the existing asphalt was replaced with new hot mix asphalt. It is 610mm wide and 102mm thick. Replacement of asphalt was done only in the southbound lane which is shown in Figure 80. This asphalt was placed to prevent any future settlement.

Figure 80 Replacement of Hot Mix Asphalt

Figure 81 Vehicle in Operation After Final Completion

Figure 81 shows the vehicle in operation. This picture was taken on July 30, 2009 which is a week after construction.

After the completion of the crosswalk repair, deflection tests and a distress survey were carried out in the crosswalk.

4.2 Repair of Sand Set Concrete Base Concrete Header at Ring Road

The temporary repair work of SSCBCH at the Ring Road was carried out on November 11, 2009 and November 12, 2009. The crosswalk was constructed on August 2007. The contractor for this work was Steed and Evans who was under warranty with the University of Waterloo to repair the contract, the crew started work at 7 am. The pavers were removed only from the faulted areas, the bedding sand was replaced in those areas and the same pavers were re-installed. The geotextile underneath was shown to be damaged and the concrete base was noted in good condition.

The failure of the crosswalk was primarily due to the drainage in the crosswalk. More specifically, water remained on this crosswalk as it was the low point of the intersection. Thus, despite having two catch basins located in the intersection, due to the elevation the water remained on the SSCBCH crosswalk. Thus, over time the crosswalk began to fail due to its continuously saturated state. A new asphalt batch was placed adjacent to the crosswalk which would result in water being drained to the catch basin areas. The drainage problem has also been fixed before the repair of the crosswalk by regarding the asphalt section in the intersection. The following Figure 82 shows work in progress. Figure 83 shows the re-graded intersection.

Figure 82 Repair work in Progress SSCBCH

Figure 83 Re-graded with Asphalt SSCBCH

Chapter 5

Performance

The overall performance of the crosswalk is analysed by different test methods. Pavement Response, Deflection test, Distress Survey and Profile are included in this section of the report.

5.1 Pavement Response

The load distribution and failure modes of flexible asphalt and interlocking concrete pavement are very similar. The pavement carries load as a flexible pavement and pavement failure primarily through rutting in the layers under the block surface. The major failure modes for the asphalt and concrete base courses are fatigue cracking, rutting and low temperature cracking.

Since the fatigue cracking is a function of horizontal strain at the bottom of asphalt, horizontal strain in concrete layers and vertical compressive stresses in granular layers, pavement response analysis included the tensile strain on the underside of the asphalt concrete base layer, tensile strain in concrete base course and the vertical compressive stress in granular subbase courses.

Trendlines of stress and strain accumulations are influenced by variations in environmental conditions i.e. temperature, rainfall and moisture.

5.1.1 Test Track

The accumulation of stress and strain in the four different crosswalk design assemblies at the Test Track in a one year period can be described as 3 million ESALs. Associated environmental data for the site has also been collected and is available. It is a significant observation that the trendlines of stress and strain accumulations are influenced by variations in environmental conditions i.e. temperature, rainfall and moisture. In short, the temperature has the strongest influence on strain.

5.1.1.1 Sand Set Concrete Base Concrete Header (SSCBCH)

Figure 84 shows the accumulation of vertical stress and moisture variation in the subbase layer in the SSCBCH from August 2007 to February 2010. It can be seen that the formation of stress is not only affected by traffic loading but also by moisture variation.

Figure 85 shows the accumulation of strain in the concrete base of the SSCBCH in the period since August 2007. It shows that the temperature variation has a direct impact on the formation of strain. High tensile strain is formed during the winter season when the temperature is below 0^0C. As the temperature increases, the tensile strain decreases in the concrete.

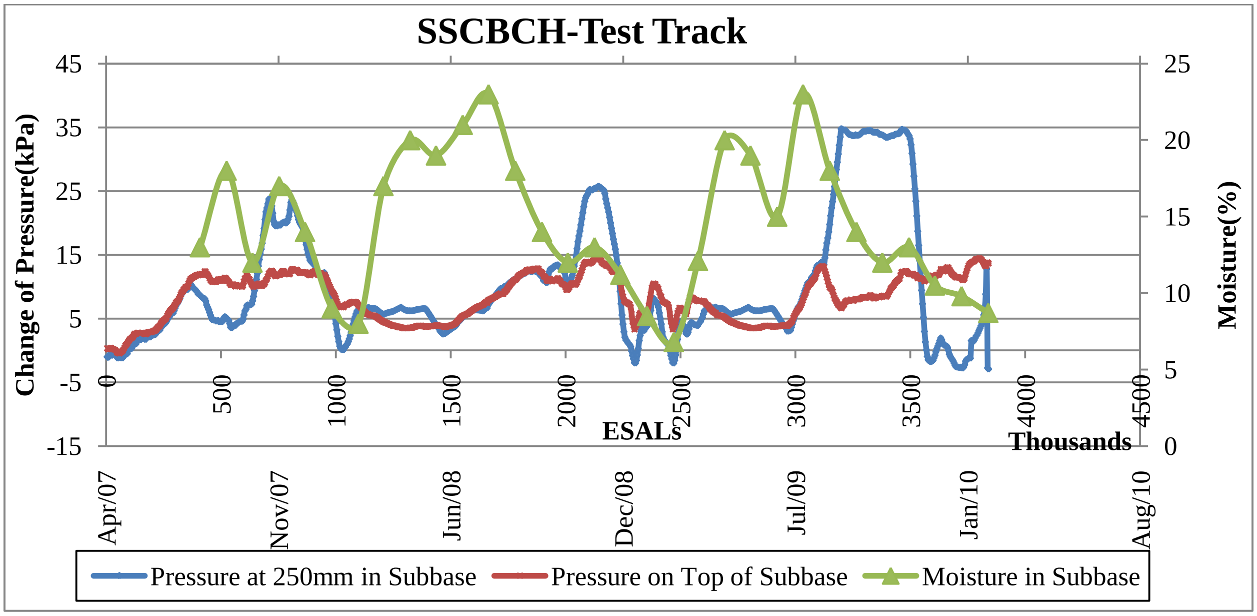


Figure 84 Measured Vertical Stress and Moisture Variation in Subbase of SSCBCH

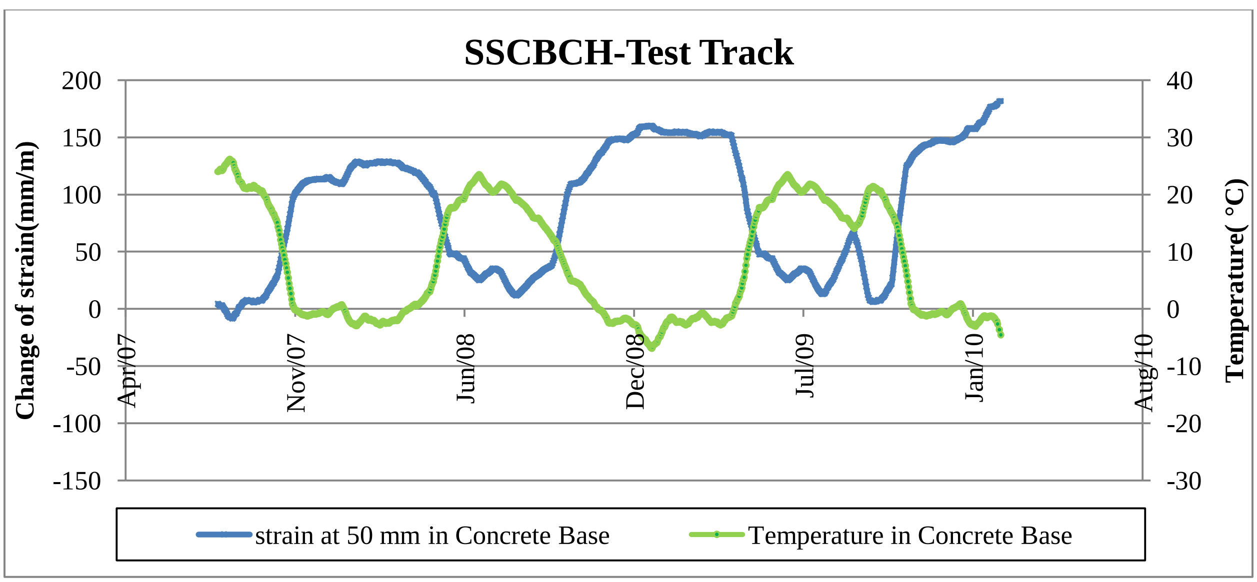


Figure 85 Accumulation of Horizontal Strain and Temperature Variation in Base of SSCBCH

5.1.1.2 Sand Set Asphalt Base Steel Header (SSABSH) and Sand Set Asphalt Base Aluminum Header (SSABAH)

SSABSH was repaired on July 2009 and was made SSABAH. However, the instrumentation was the same. The line indicates after and before the repair work. Figure 86 shows the accumulation of vertical stress and moisture variation in the subbase layer of the SSABSH during the period from August 2007 to February 2010. It can be seen that the formation of stress is affected by moisture variation. Tensile vertical stress is observed at the top of the subbase as well as at 250 mm below in the subbase. The maximum amount of tensile stress is noted in September 2008. The earth pressure cell at the 250 mm in subbase stopped functioning after June 2008 which is why the data is not showing up in the figure.

Figure 87 shows the accumulation of strain at the bottom of the asphalt layer in the SSABSH in this period. It can be seen that temperature has a direct impact on the formation of strain. High compressive strains are formed during winter when the temperature is below 0^0C. As the temperature increases, the tensile strain increases at the bottom of the asphalt base layer.

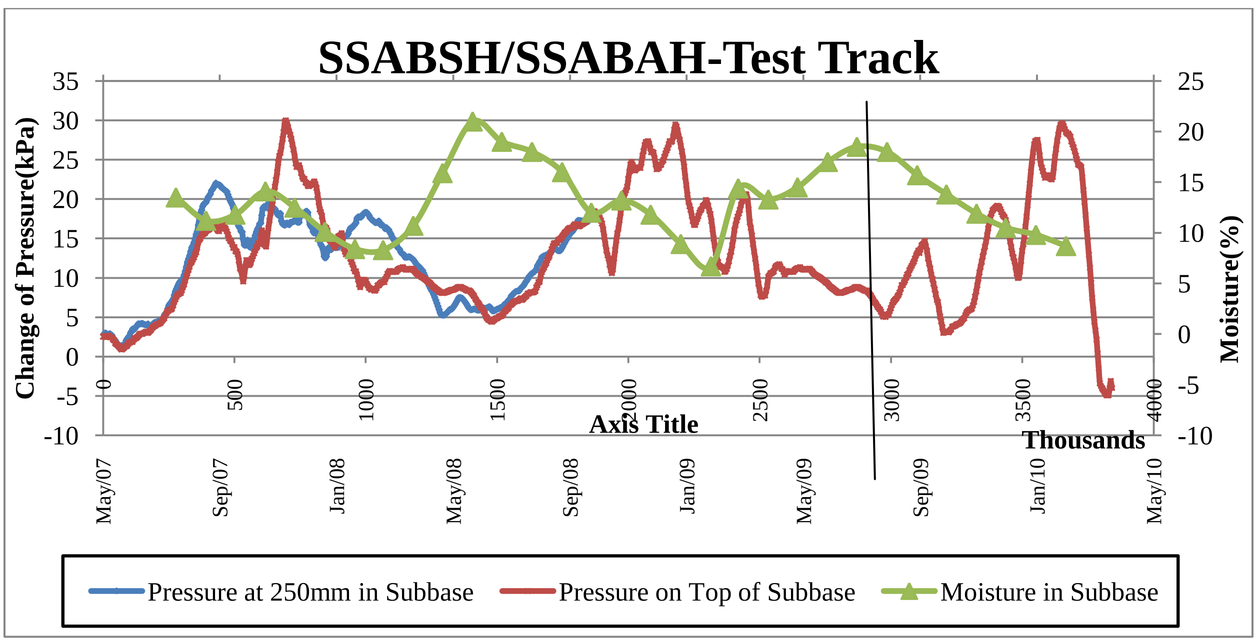


Figure 86 Accumulated Vertical Stress and Moisture Variation in Subbase of SSABSH/SSABAH

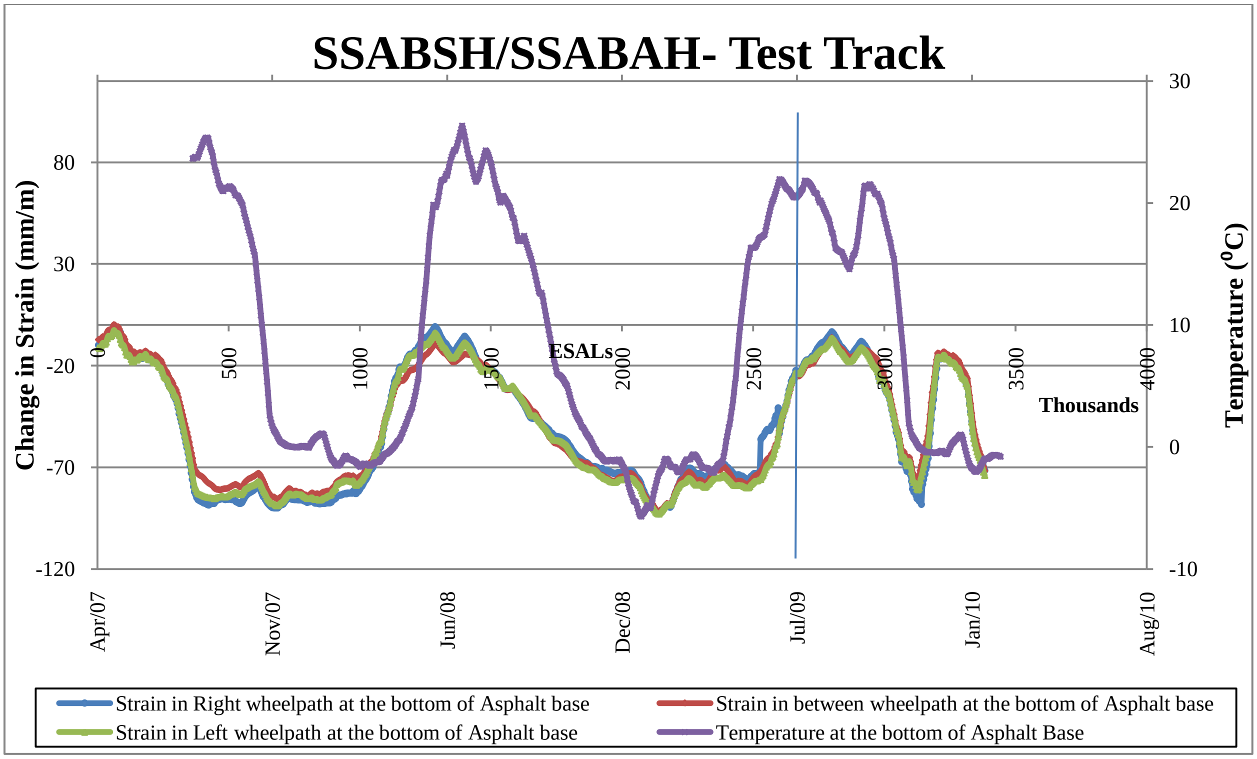


Figure 87 Accumulated Horizontal Strain and Temperature Variation in Base of SSABSH/SSABAH

5.1.1.3 Bituminous Set Concrete Base Concrete Header (BSCBCH)

Figure 88 shows the accumulation of vertical stress and moisture variation in subbase layer in BSCBCH. It can be seen that the formation of stress is affected by moisture variation. Tensile vertical stress is formed on the top of the subbase as well as at 250 mm below the subbase. The figure indicates that tensile stresses increases as moisture content increases.

Figure 89 shows the accumulation of strain in the concrete base of the BSCBCH crosswalk section from August 2007 to February 2010. It can be seen that temperature has a direct impact on the formation of strain. High tensile strain is formed during winter when the temperature is below 0^0C. As the temperature increases the tensile strain decreases in the concrete.

At the Test Track, when the three sections are compared as shown in Figures 90 and 91, the stresses and strains are observed as follows: the maximum observed stress for all sections is measured in the SSCBCH followed by the SSABAH and the BSCBCH crosswalks. The maximum strain is observed in the BSCBCH followed by the SSCBCH and SSABSH crosswalks.

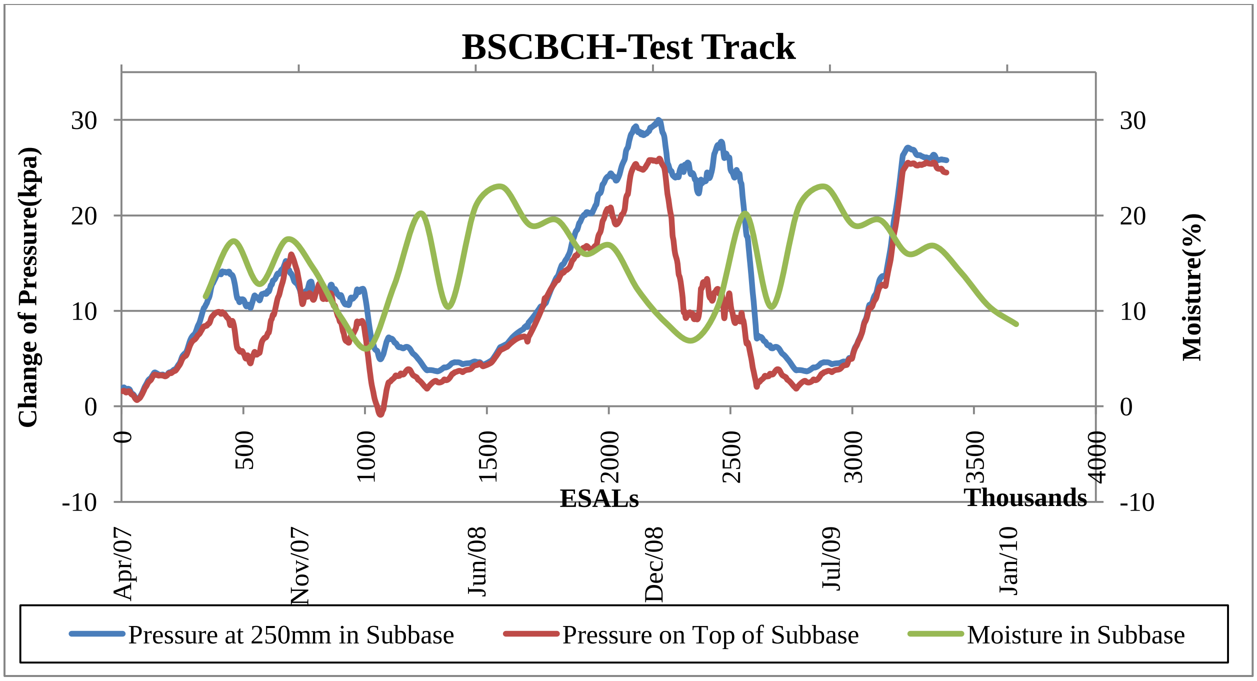


Figure 88 Accumulation of Vertical Stress in Subbase of BSCBCH

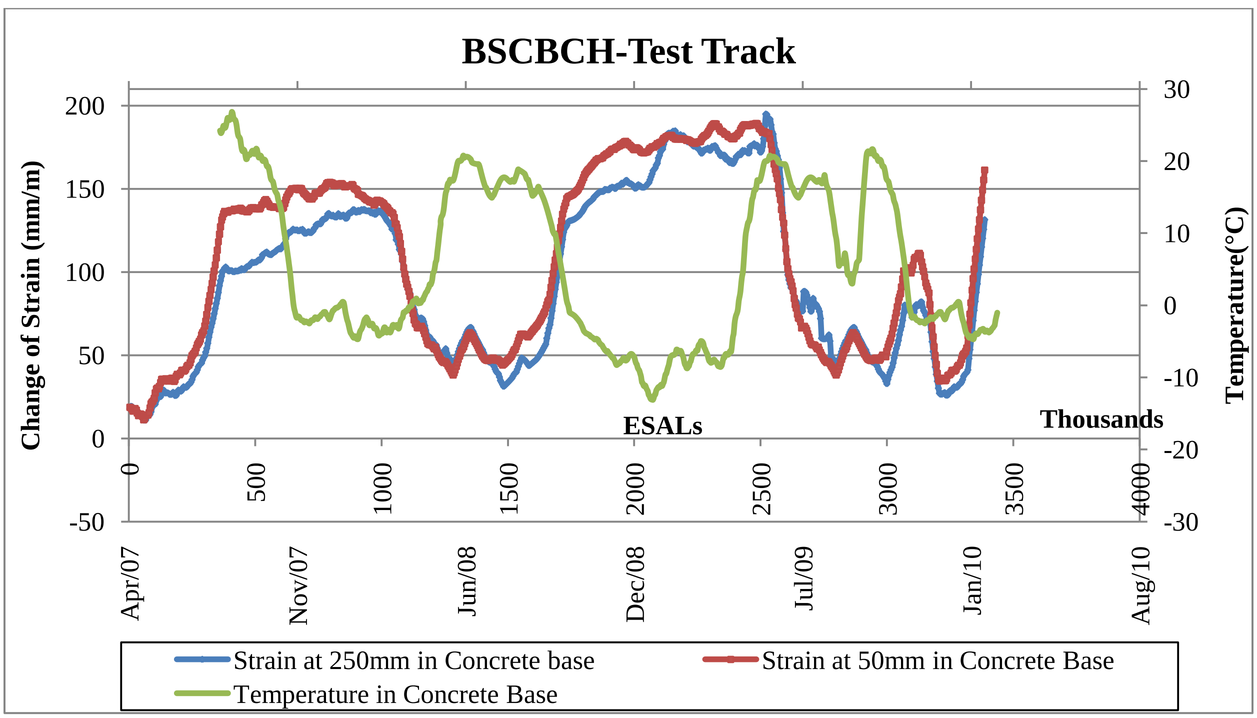


Figure 89 Accumulation of Horizontal Strain in Base of BSCBCH

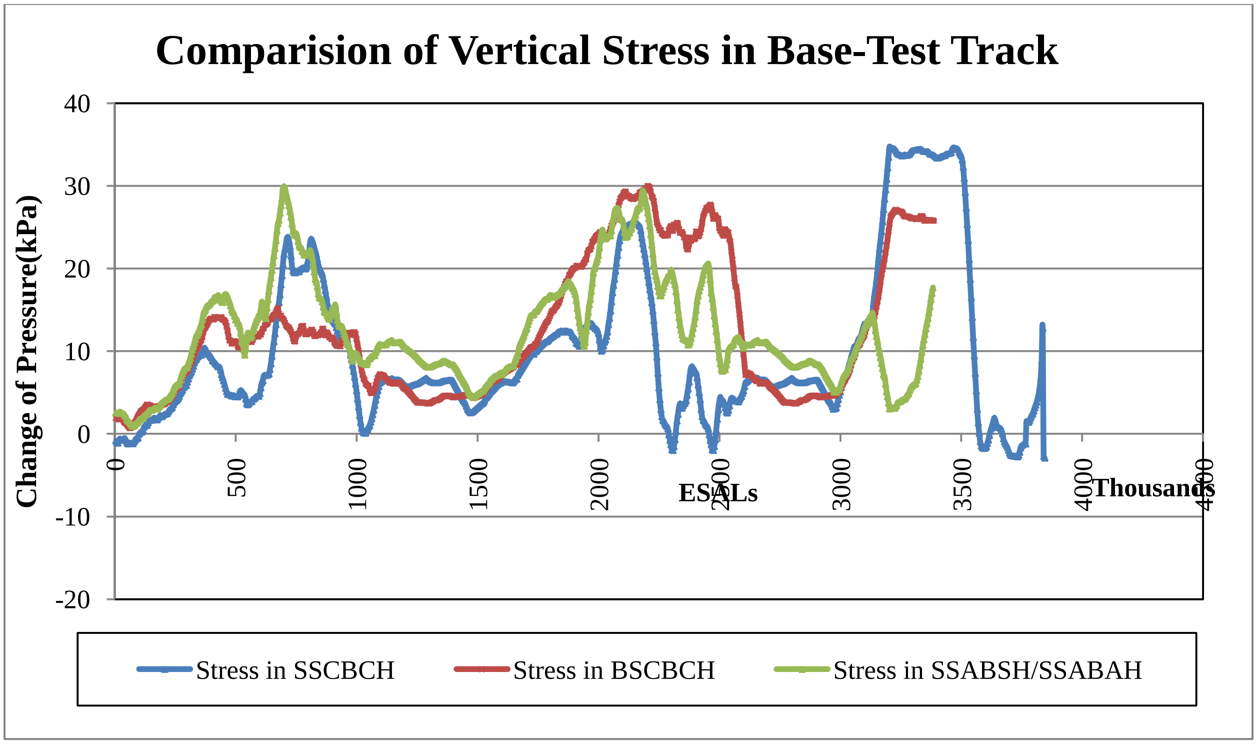


Figure 90 Accumulated Vertical Stress in Subbase in All Crosswalks

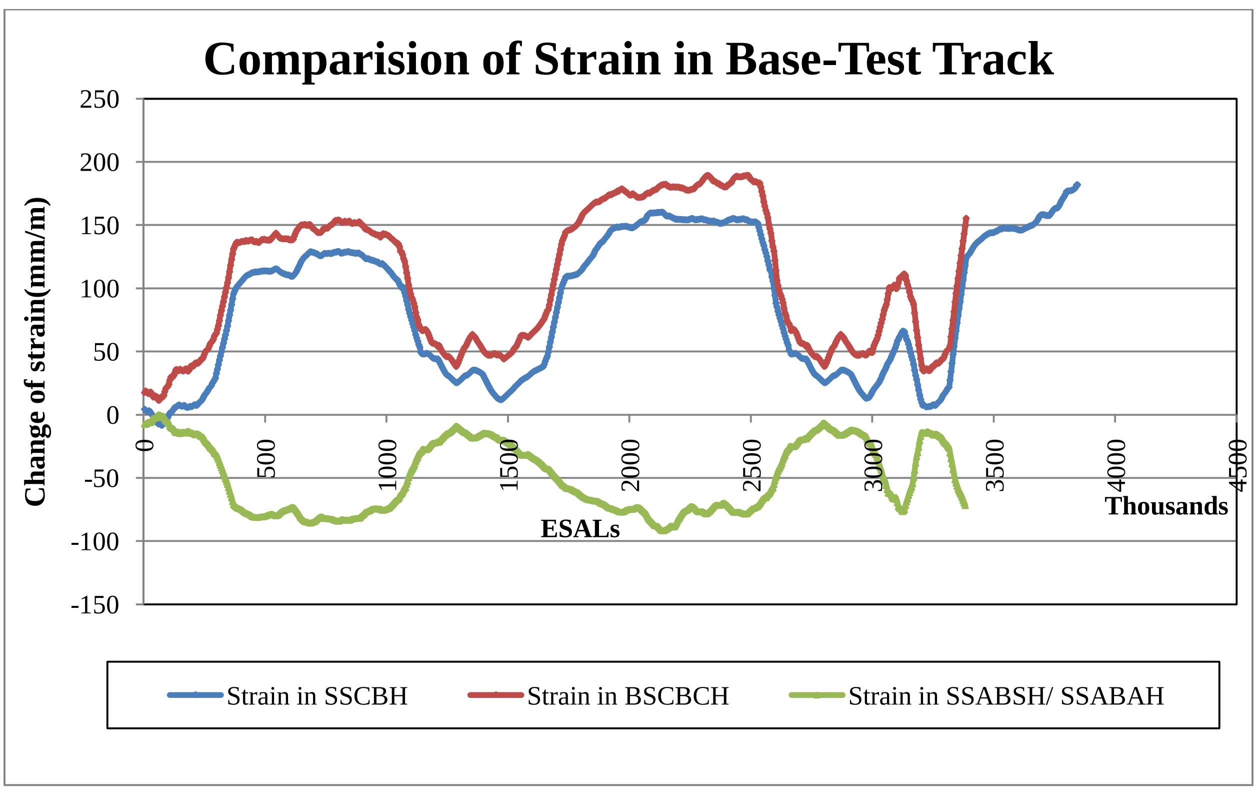


Figure 91 Accumulated Horizontal Strain in Base in All Crosswalks

5.1.1.4 Sand Set Granular Base Aluminum Header (SSGBAH)

Figure 92 shows the moisture and temperature variation in the subbase during a six month period from September 2009 to February 2010. The temperature at the different depths of the subbase does not show a significant difference while the moisture variation can be seen significantly in the two different depths of the Granular base crosswalk. The moisture has dropped below 0°C during the first week of October at 375mm from the top of the subbase.

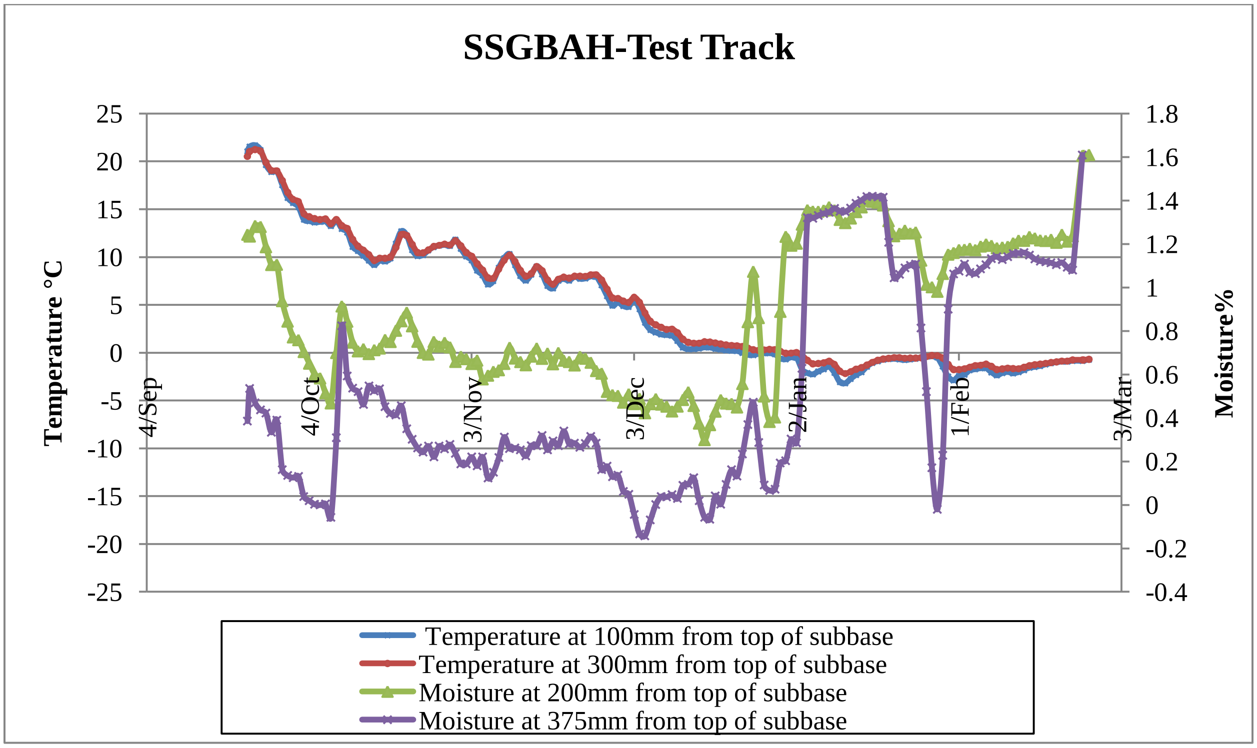


Figure 92 Moisture and Temperature Variation in the Subbase of SSGBAH

5.1.2 Ring Road

The accumulation of strain in the three different crosswalk design assemblies at the UW Ring Road over twenty eight months period (2,90,000 ESALs) and the respective environmental data are presented. Trendlines of strain accumulations are influenced by variations in environmental conditions i.e. temperature, rainfall and moisture.

5.1.2.1 Sand Set Concrete Base Concrete Header (SSCBCH)

Figure 93 shows the accumulation of strain in the concrete base layer of SSCBCH at the Ring Road from November 2007 to February 2010. It can be seen that temperature has direct impact on the formation of strain. High tensile strain is formed during the winter months when the temperature is below 0^0C. As the temperature increases, the tensile strain decreases and the compressive strain increases. The compressive strain reached 160 microstrains when the temperature was recorded to be higher than 30^{0}C. The tensile strain reached 82 microstrains when the temperature was just -7°C in January 2009.

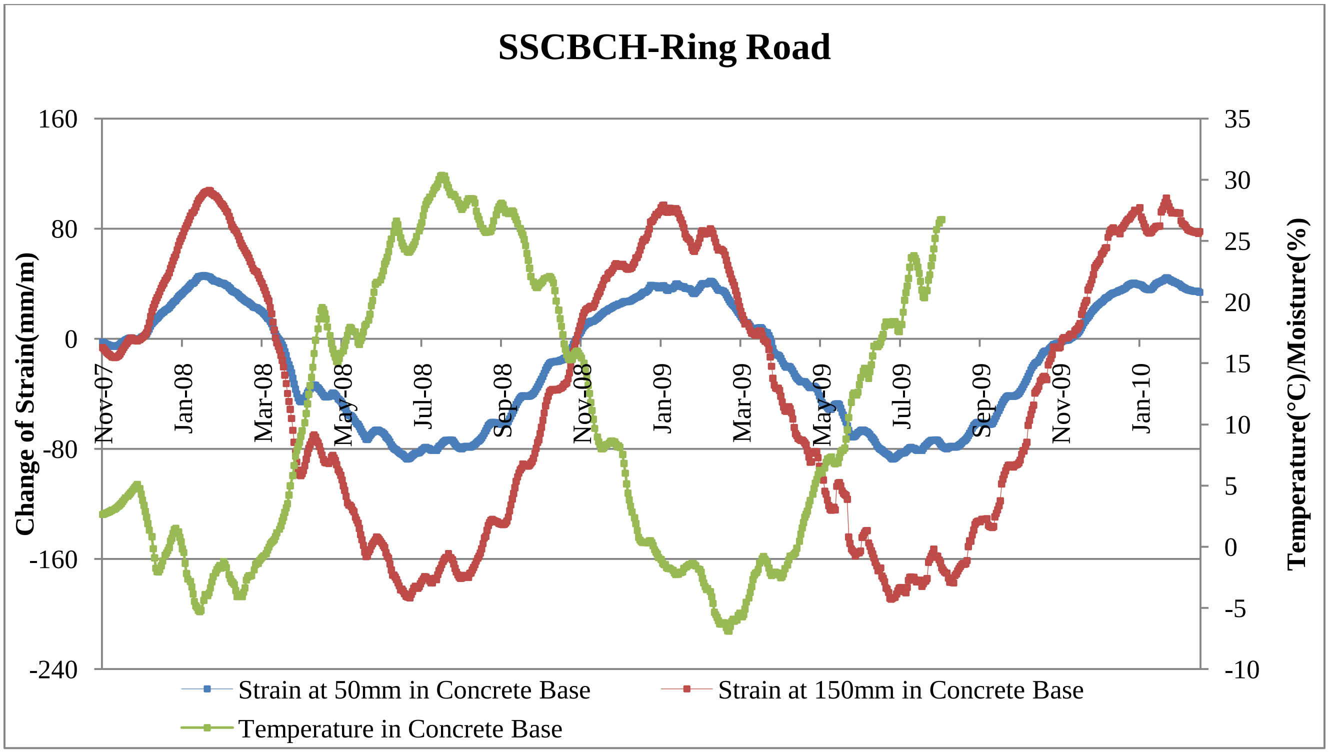


Figure 93 Accumulated Horizontal Strain and Temperature and Moisture Variation in SSCBCH

5.1.2.2 Bituminous Set Concrete Base Concrete Header (BSCBCH)

Figure 94 shows the accumulation of strain in the concrete base layer of the BSCBCH in the period of twenty eight months since November 20, 2007. It can be seen that temperature has direct impact on the formation of strain. High tensile strain is formed during winter when the temperature is below 0^{0}C. As temperature increases the tensile strain decreases and the compressive strain increases.

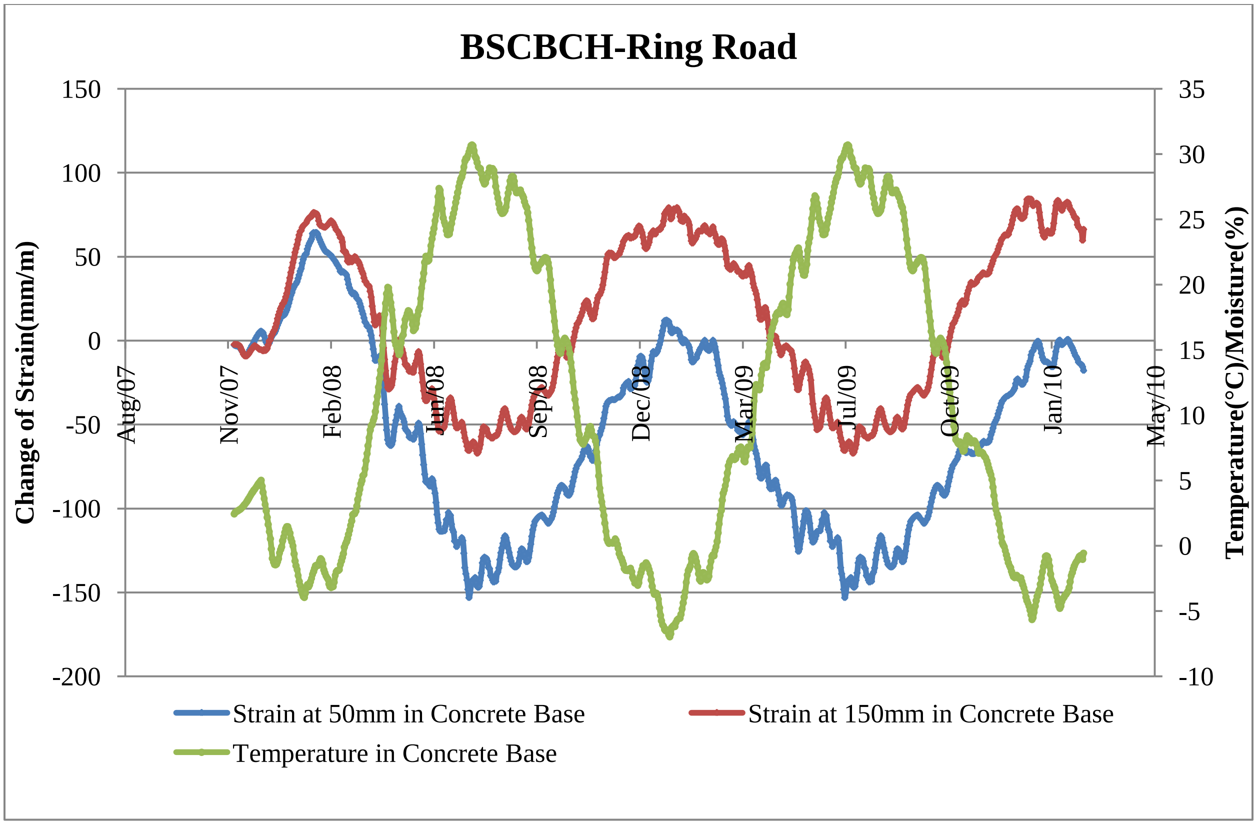


Figure 94 Accumulated Horizontal Strain and Temperature and Moisture Variation in BSCBCH

5.1.2.3 Sand Set Asphalt Base Aluminum Header (SSABAH)

Figure 95 shows the accumulation of strain at the bottom of the asphalt base layer of the SSABAH from November 2007 to February 2010. It can be seen that the temperature has the direct impact on the formation of strain. High compressive strain is formed during the winter when the temperature is below 0^{o}C. Tensile strain forms as the temperature increases.

Figure 96 shows the strain accumulation in all test sections at the Ring Road. The maximum tensile strain is observed in the SSABAH crosswalk while the maximum compressive strain is observed in BSCBCH during this period.

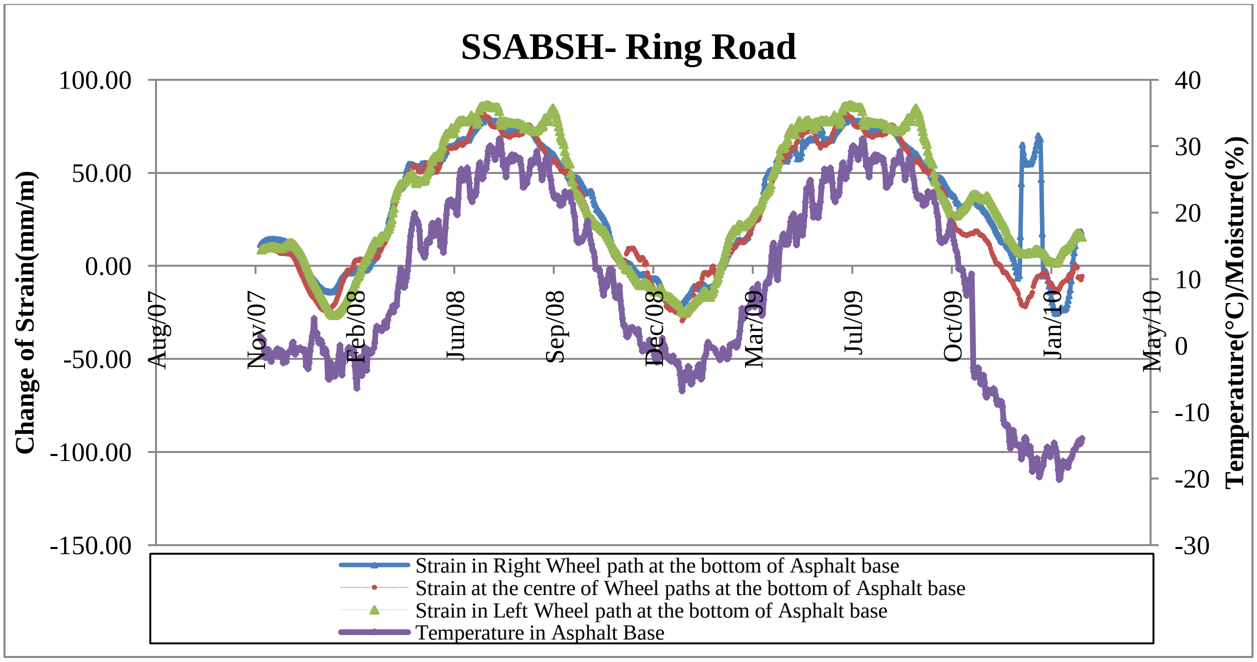


Figure 95 Accumulated Horizontal Strain and Temperature and Moisture Variation in SSABAH

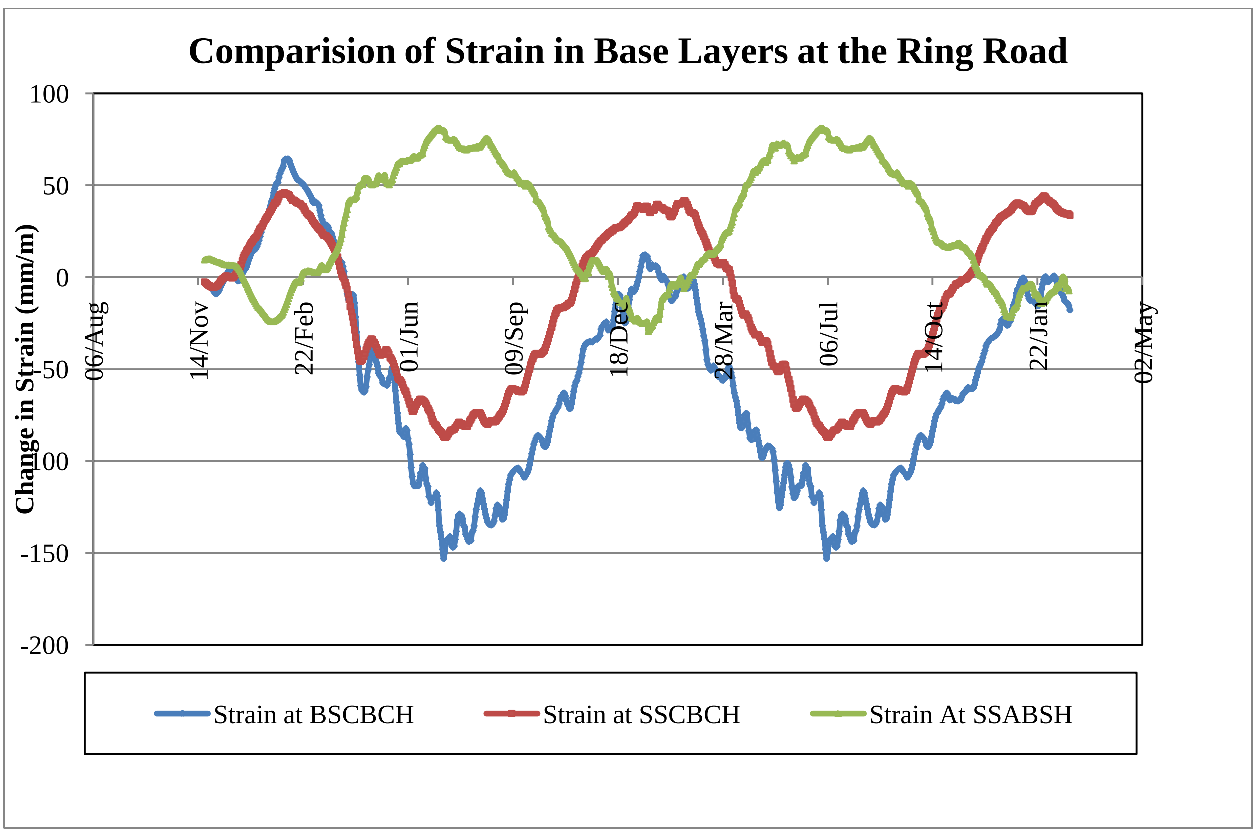


Figure 96 Accumulated Horizontal Strain in All Crosswalk Sections

5.1.3 PREDICTIVE MODELING

Predictive modelling is the process by which a model is created to try to best predict the probability of an outcome. Predictive models for the strain sensors are created in this report.

5.1.3.1 Strain

There are fourteen strain sensors in the crosswalks, out of them one at SSCBCH at Test Track is not working. The predictive models for all other strain sensors are created to predict the raw strain values recorded by the data logger. The strain for each crosswalk was best described with second degree polynomial equation. The prediction ability of the strain equation is shown in the Figure 97 to Figure 100 below at 50mm and 150mm from concrete base. BSCBCH crosswalks at both the locations are chosen as the sample.

5.1.3.1.1 Test Track

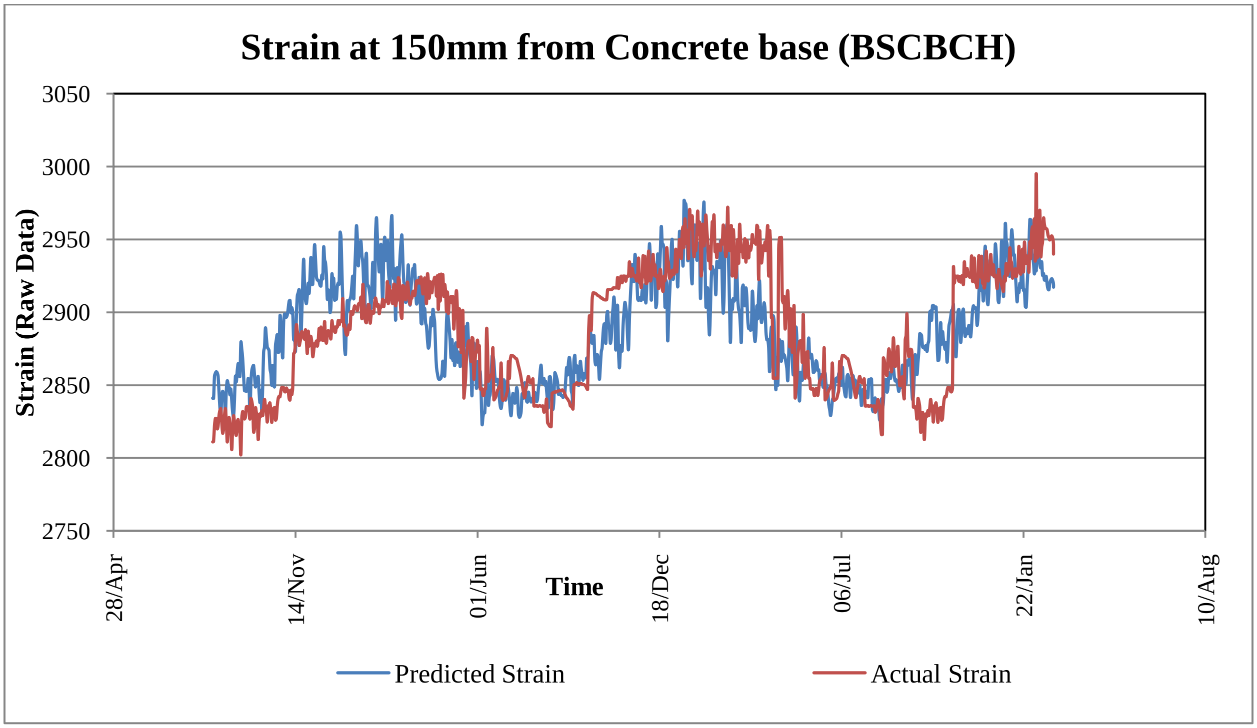


Figure 97 Actual Vs Predicted Strain at 150mm from Concrete base (BSCBCH, Test Track)

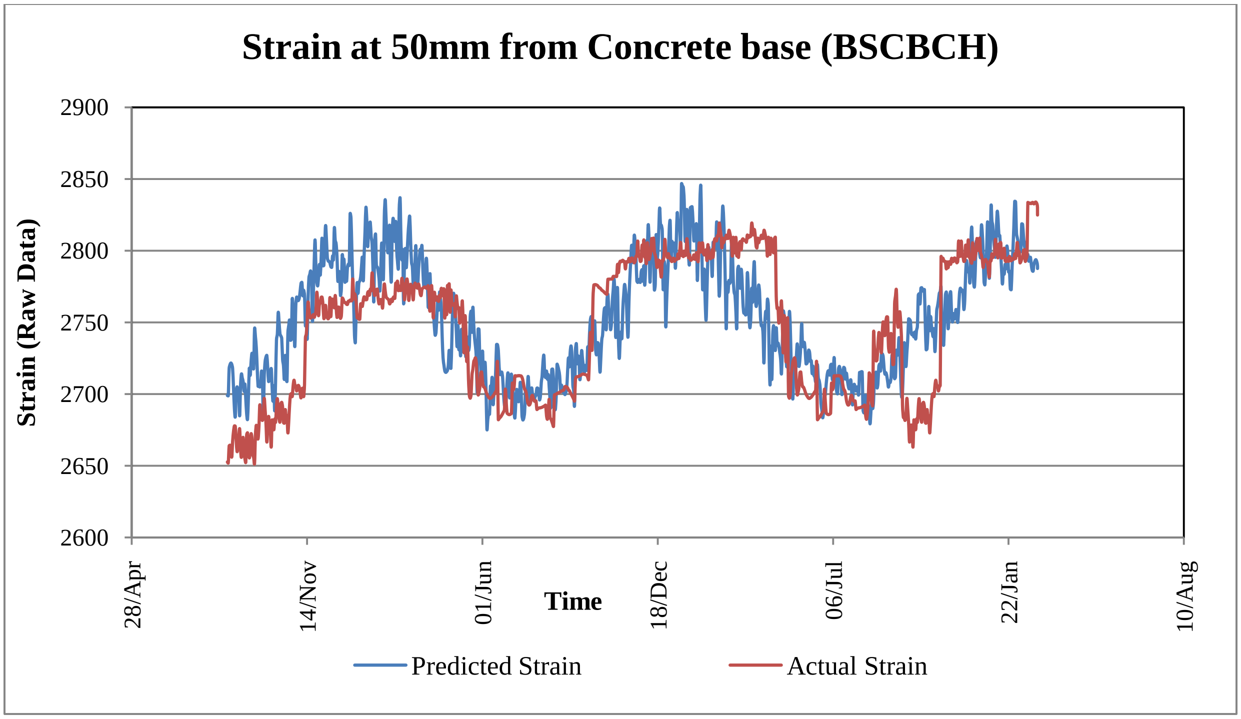


Figure 98 Actual Vs Predicted Strain at 50mm from Concrete base (BSCBCH, Test Track)

5.1.3.1.2 Ring Road

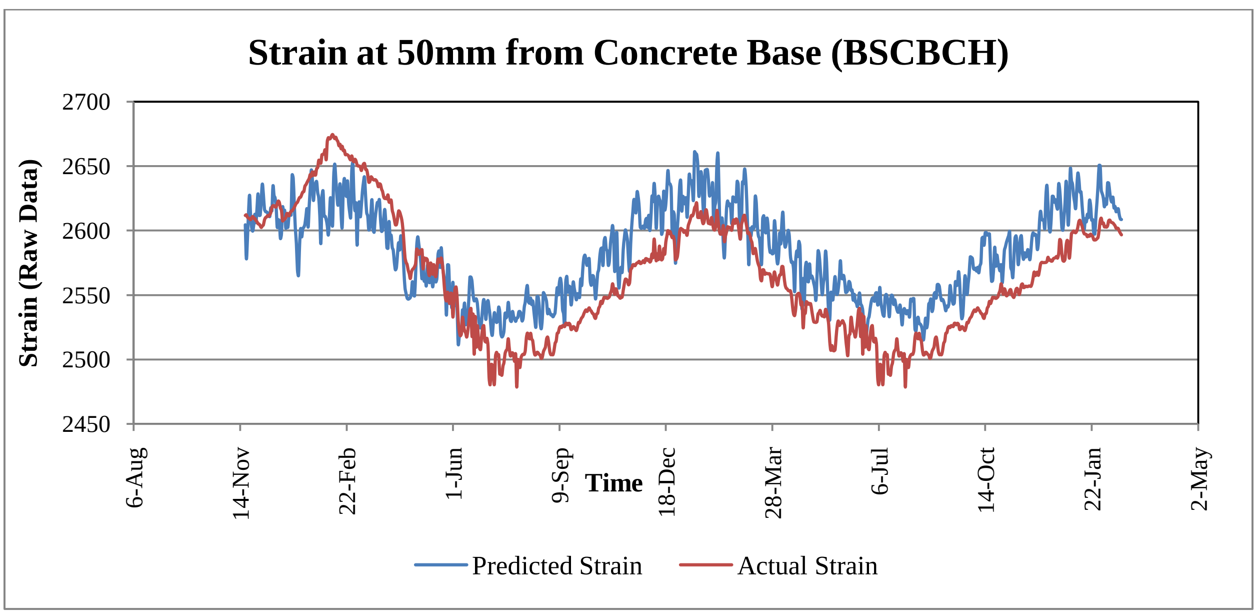


Figure 99 Actual Vs Predicted Strain at 50mm from Concrete base (BSCBCH, Ring Road)

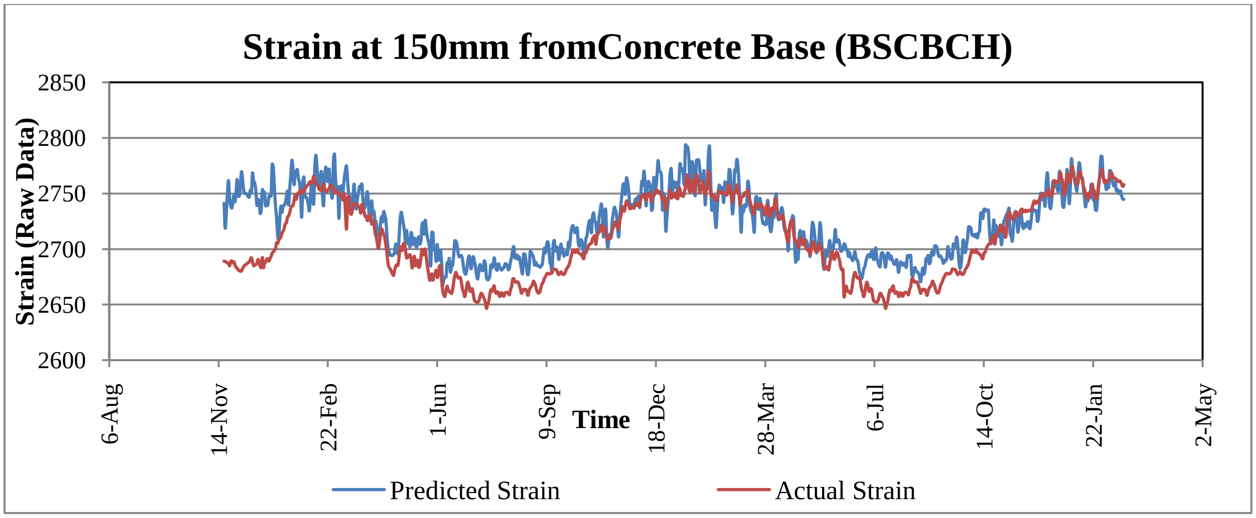


Figure 100 Actual Vs Predicted Strain at 150mm from Concrete base (BSCBCH, Ring Road)

From the prediction samples, the models on the sensors 150mm from the concrete base have more predictive ability then the sensors on the upper part which is 50mm from the concrete base. Table 10 represents the predictive strain equations for each working sensors. The R^2 value is near to 1, which shows the good correlation between the predictive model and the actual sensor reading.

Table 10 Predictive Strain Equations

Project Location	Sensor		Predictive Raw Strain Equation	R^2
	Crosswalk	Location		
Test Track	BSCBCH	50mm from Concrete base	$\mu\varepsilon = -0.0058T^2 - 3.7766T + 2779.1$	0.8516
		150mm from Concrete base	$\mu\varepsilon = 0.0057\ T^2 - 3.3952\ T + 2906.9$	0.8076
	SSCBCH	50mm from Concrete base	$\mu\varepsilon = 0.0282\ T^2 - 4.6643\ T + 2802.3$	0.8633
	SSABAH	Strain in Right wheel path at the bottom of Asphalt base	$\mu\varepsilon = 0.0352\ T^2 + 2.9743\ T + 2543.2$	0.8685
		Strain in between wheel path at the bottom of Asphalt base	$\mu\varepsilon = 0.03\ T^2 + 3.0008\ T + 2496.4$	0.9482
		Strain in Left wheel path at the bottom of Asphalt base	$\mu\varepsilon = 0.0246\ T^2 + 3.2034\ T + 2566.2$	0.9342
Ring Road	BSCBCH	50mm from Concrete base	$\mu\varepsilon = -0.0064\ T^2 - 3.2819\ T + 2602.8$	0.7696
		150mm from Concrete base	$\mu\varepsilon = 0.005\ T^2 - 2.8476\ T + 2739.7$	0.8575
	SSCBCH	50mm from Concrete base	$\mu\varepsilon = 0.0017\ T^2 - 2.7542\ T + 2650.3$	0.9479
		150mm from Concrete base	$\mu\varepsilon = 0.0219\ T^2 - 3.7866\ T + 2733.6$	0.9368
	SSABAH	Strain in Right Wheel path at the bottom of Asphalt base	$\mu\varepsilon = 0.0561\ T^2 + 2.0313\ T + 2585.8$	0.6165
		Strain in Left Wheel path at the bottom of Asphalt base	$\mu\varepsilon = 0.0629\ T^2 + 2.2542\ T + 2546.7$	0.8668

5.2 Deflection Testing

5.2.1 Portable Falling Weight Deflectometer (PFWD)

Deflection measurement techniques are widely used for the structural evaluation of pavement. Portable Falling Weight Deflectometer (PFWD) was used in the deflection testing of the crosswalk section. The PFWD is a dynamic impact type device. In order to simulate a load impulse similar to traffic loading, a weight is dropped on a loading plate in contact with the road. The deflection is measured by geophones located at different distances from the loading system. For this study, the CPATT Dynatest 3031 Light Weight Deflectometer (LWD) has been selected. The LWD is a light, portable device that creates a non-destructive shock-wave through the soil as a result of the impact of a falling mass (10, 15, or 20 kg) from a variable drop height (100 mm to 850 mm). The impact force is transmitted to the underlying surface through a 100, 200 or 300 mm diameter loading plate. Figure 101 shows the different parts of the Dynatest 3031. The LWD 3031 records force, pressure, elastic modulus and deflection with respect to time which are stored automatically in a Personal Digital Assistant (PDA). A PDA equipped with the LWD 3031 software is used to record the stress and deflection that are measured by the sensors. The program calculates the surface elastic modulus using the following equation-3:

$$E_0 = \frac{f \cdot (1-\nu^2) \cdot \sigma_0 \cdot a}{d_0} \qquad \text{(Equation-3)}$$

Where

f is a factor that depends on the stress distribution,

v is the Poisson's ratio of the material,

σ_0 is the applied stress at surface,

a is the radius of the loading plate, and

d_0 is the centre deflection.

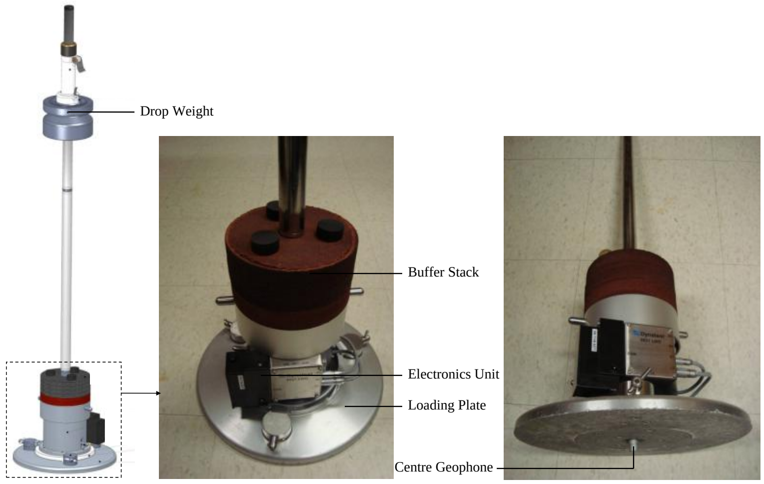


Figure 101 CPATT Dynatest LWD 3031

The measurements were taken utilizing a 20 kg drop weight and a 300 mm loading plate. In all cases, six good measurements were taken and the average of six was used for analysis and comparison. The structural behaviour of each section is compared in terms of composite elastic modulus, Deflection.

5.2.1.1 Test Track

At the Test Track, five test locations were selected per section. The test points are located in two wheel paths, at the edge of pavement and at the center line of the crosswalk sections as shown in Figure 102.

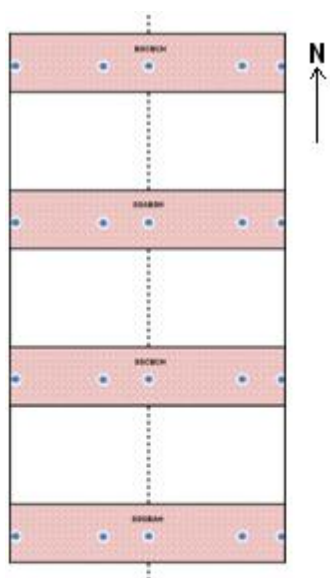


Figure 102 Test Points of LWD

All sections are compared in terms of the average deflection and the elastic modulus in Figure 103 and Figure 104 respectively. The maximum deflection is observed in the SSGBAH (459 microns) followed by

SSCBCH (286 microns), the SSABAH/SSABSH (137 microns) and the BSCBCH (31 microns). The average deflection of November 2008, July 2009, September 2009, November 2009 and April 2009 is shown in Figure 105. From the Figure, it can be noted that, after the repair of the SSABAH section, the deflection has decreased. The deflection results on November 2009 are slightly less than the deflection in July 2009. This is due to the variable temperature conditions and it is notable that in July it was very wet and this high moisture could be leading to higher deflection values. And the deflection results of November 2009 and April 2009 are near. The data is tabulated in Table 11.

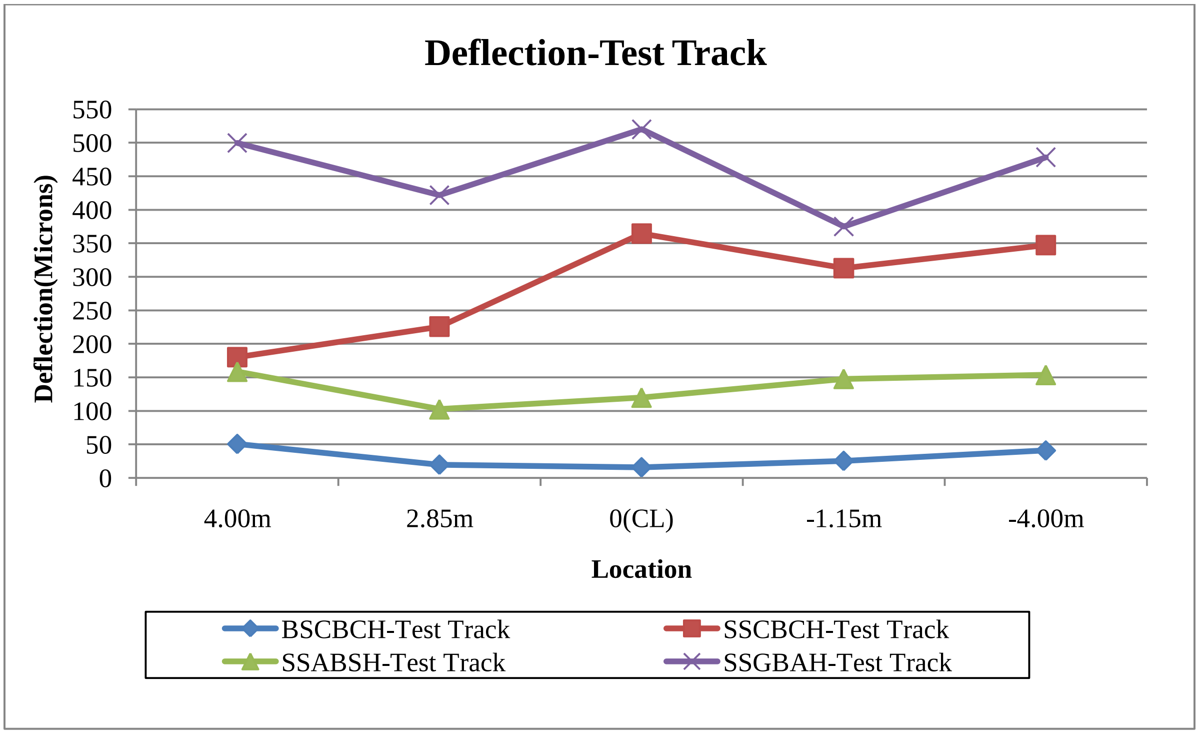


Figure 103 Deflections observed at the Test Track in all Crosswalks.

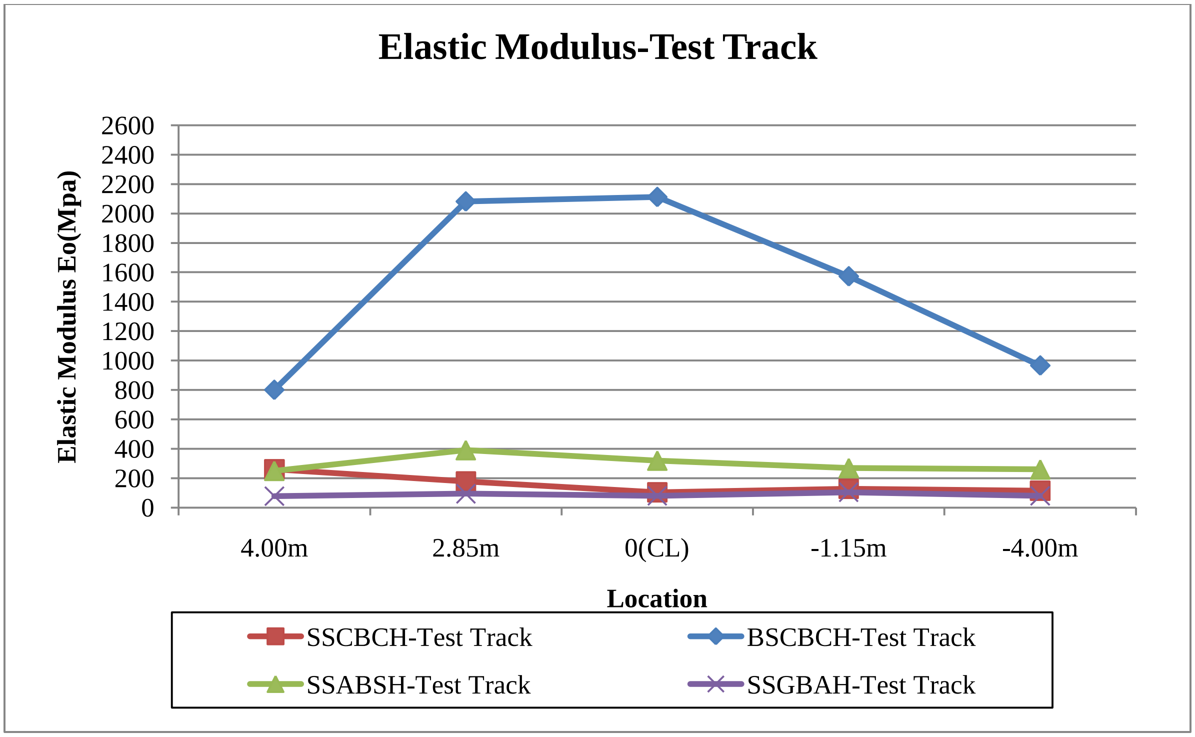


Figure 104 Elastic Modulus observed at the Test Track in all Crosswalks

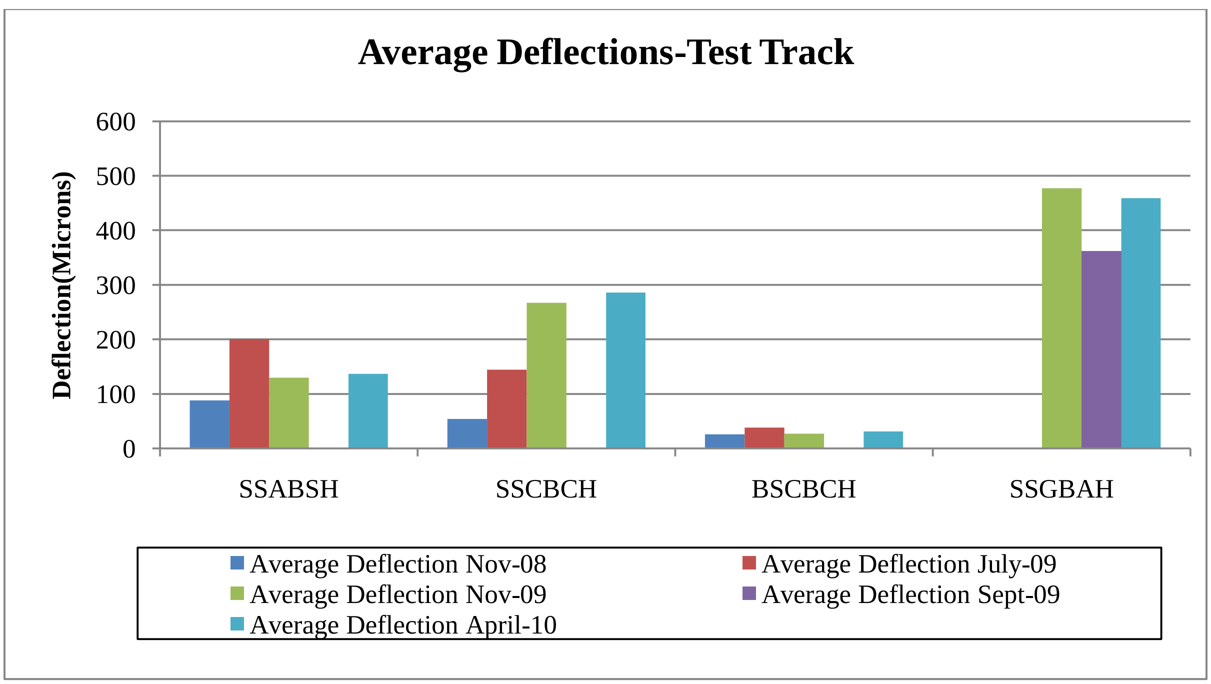


Figure 105 Average Deflection at the Test Track

Table 11 Tabular Representation of Average Deflection Over Time in Test Track

Test Track-CW	Average Deflection Nov-08	Average Deflection July-09	Average Deflection Nov-09	Average Deflection Sept-09	Average Deflection April-10	Mean	Standard Deviation
SSABSH	88	200	129	-	137	139	47
SSCBCH	54	144	267	-	286	188	109
BSCBCH	26	38	27	-	31	31	6
SSGBAH	-	-	477	362	459	433	62

Table 11 shows the standard deviation of the deflection values over time. A low standard deviation indicates that the data points tend to be very close to the mean, whereas high standard deviation indicates that the data are spread out over a large range of values. At the Test Track, the standard deviation of the BSCBCH crosswalk is lowest which shows that data points are close to the mean which is 31. The standard deviation of the SSCBCH crosswalk is very high, average deflection of November 2008 is 54 while the mean is 188.

5.2.1.2 Ring Road

At the Ring road, the test points were strategically selected to measure the deflection. For this reason the SSCBCH has five test sections in the wheel paths, at the edge of pavement and at the center line of the crosswalk, the BSCBCH has four test sections which does not include centre of the crosswalk due to the geometrics in that particular crosswalk, and the SSABAH and the SSGBCH has six test sections; one additional test point in the bus bay area. The schematic description of test points is provided in Figure 106.

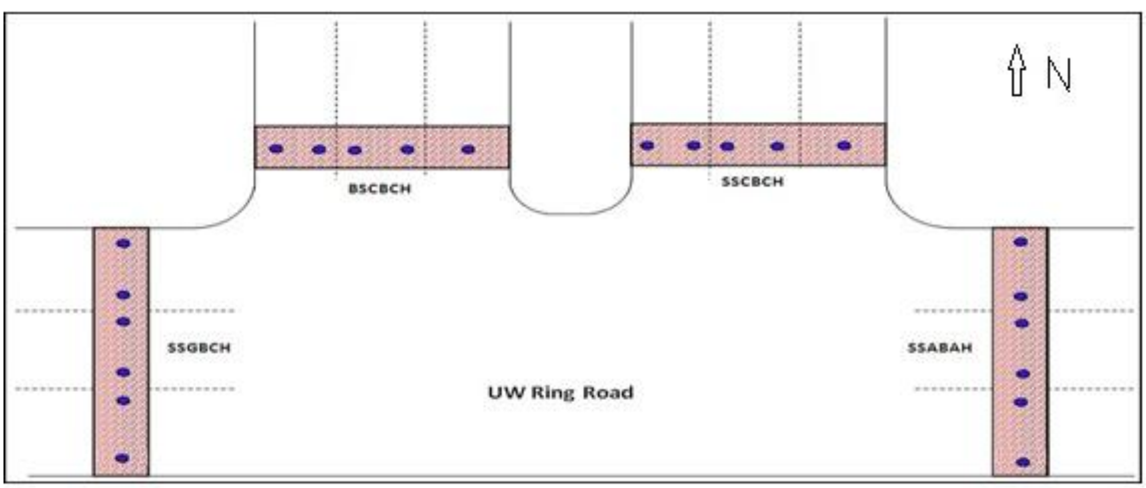


Figure 106 Test Points of LWD at Ring Road

The maximum deflection is observed in the SSGBCH crosswalk (206 microns) followed by the SSABAH crosswalk (162 microns), the SSCBCH crosswalk (100 microns) and the BSCBCH crosswalk (28 microns). All sections are compared in terms of elastic modulus and average deflection from Figure 107 to Figure 110. It was inconvenient to show all the crosswalk section of the Ring Road in one plot due to differing test sections; so it is provided in separate graphs.

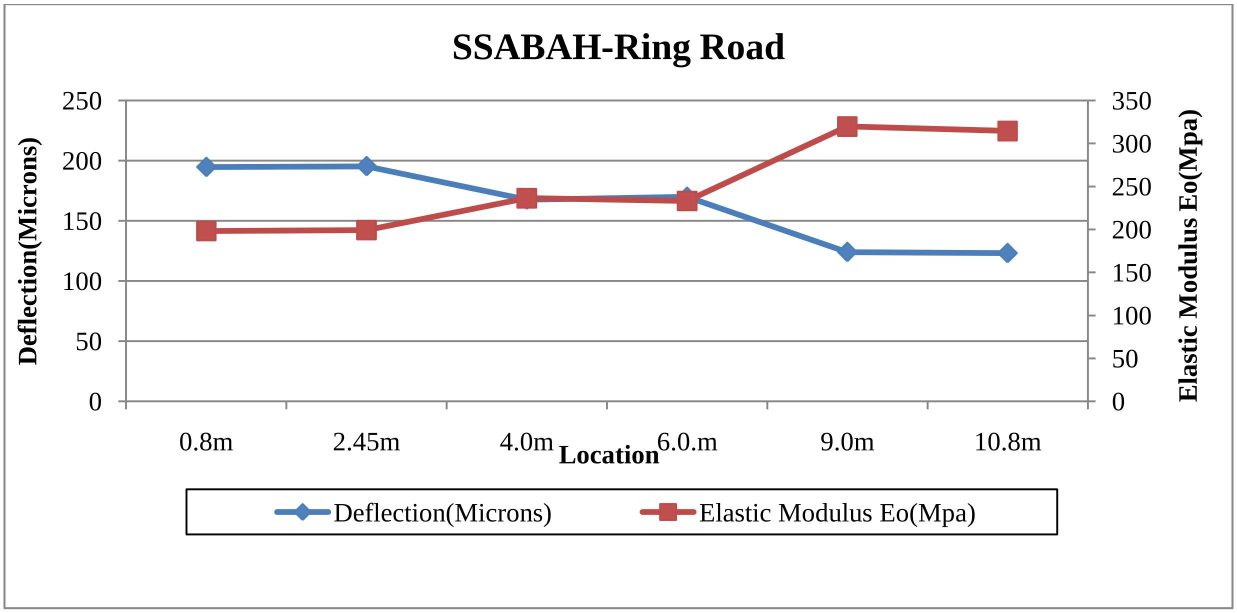


Figure 107 Elastic Modulus and Deflection in SSABAH at the Ring Road

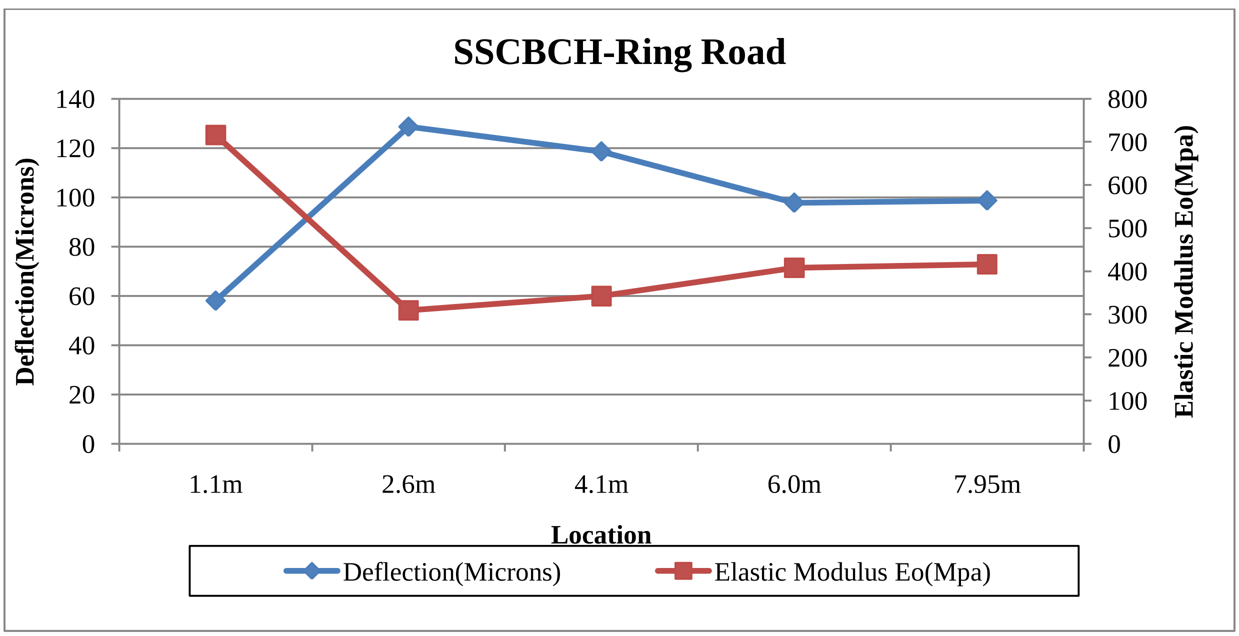


Figure 108 Elastic Modulus and Deflection in SSCBCH at the Ring Road

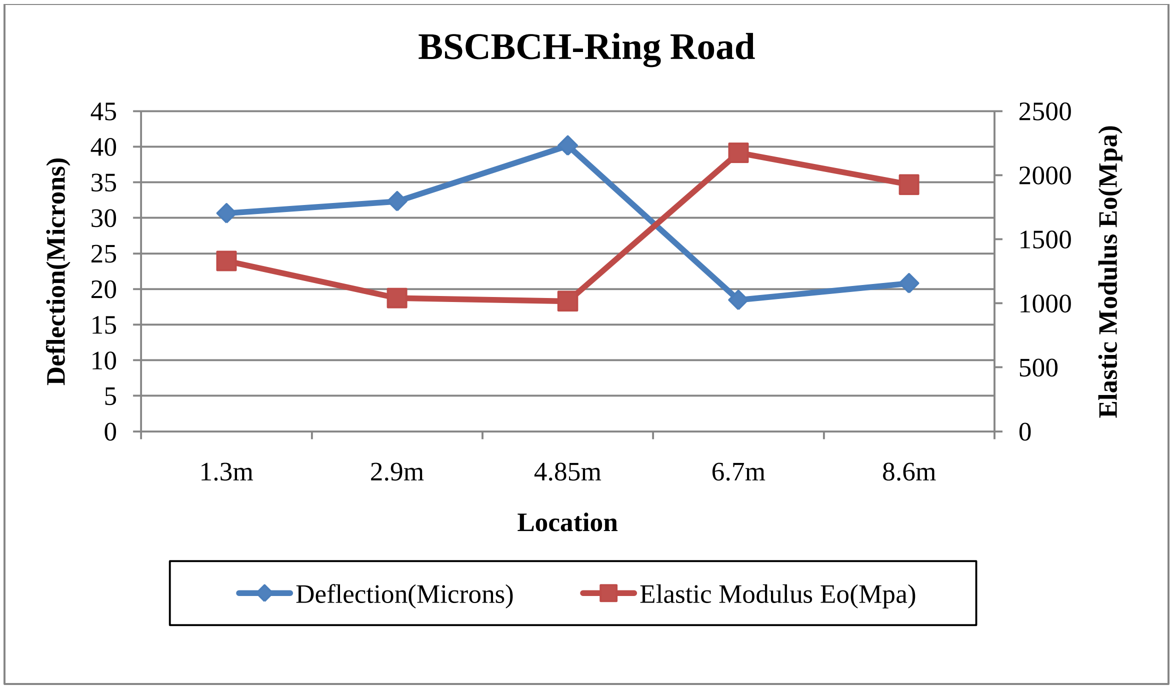


Figure 109 Elastic Modulus and Deflection in BCCBCH at the Ring Road

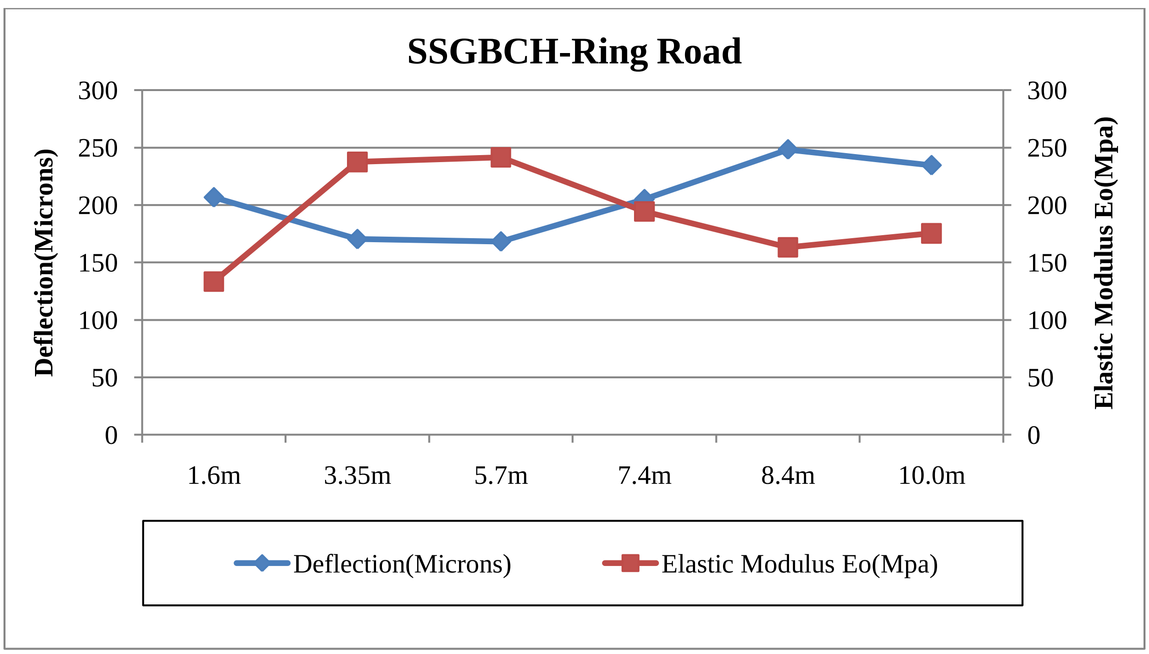


Figure 110 Elastic Modulus and Deflection in SSGBCH at the Ring Road

The average deflection in November 2008, July 2009, November 2009 and April 2010 are shown in Figure 111. The deflection results in November 2009 are slightly less than the deflection in July 2009. This is due to the variable temperature conditions and other factors. The deflection result for SSABAH in April 2010 is higher than in November 2009. The data is tabulated in Table 12.

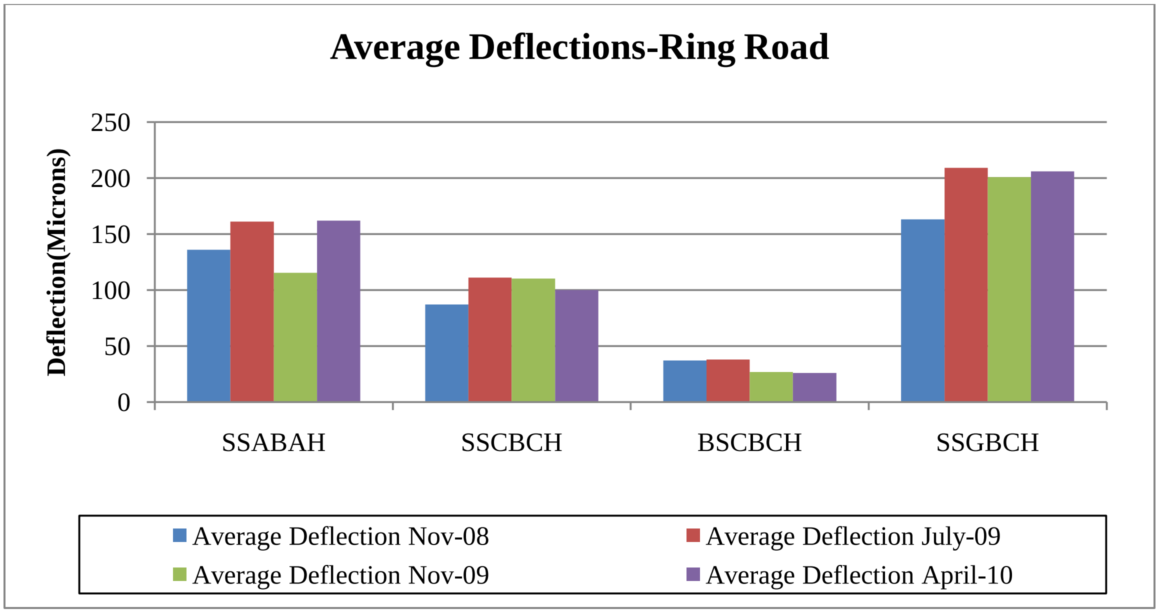


Figure 111 Average Deflection at the Ring Road

Table 12 Tabular Representation of Average Deflection Over Time in Ring Road

Ring Road-CW	Average Deflection Nov-08	Average Deflection July-09	Average Deflection Nov-09	Average Deflection April-10	Mean	Standard Deviation
SSABAH	136	161	115	162	144	22
SSCBCH	87	111	110	100	102	11
BSCBCH	37	38	27	26	32	6
SSGBCH	163	209	201	206	195	21

Table 12 also shows the standard deviation of the deflection values over time at the Ring Road. A low standard deviation indicates that the data points tend to be very close to the mean, whereas high standard deviation indicates that the data are spread out over a large range of values. At the Ring Road, the standard deviation of the BSCBCH crosswalk is lowest which shows that data points are close to the mean which is 32. The standard deviation of the all other crosswalk at Ring Road is less in comparison to the deviation observed in the Test Track.

5.2.2 Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is a non-destructive deflection testing device for use in pavement structural evaluation. The FWD is a readily available, industry accepted testing instrument that measures the pavement response (i.e., deflection) to a load that simulates the in-service truck loads applied to the pavement. The FWD applies a dynamic load through a circular plate that is lowered to the pavement surface. Sensors in contact with the surface measure the downward deflection of the pavement surface. The use of a FWD enables the engineer to determine a deflection basin caused by a controlled load.

FWD testing was carried out in the CPATT Test Track-Crosswalks on September 16, 2009 with the help of Stantec Consulting Limited. Six drops were carried out at the edge of the crosswalk with three different loadings. Figure 112 shows the test being done in Sand Set Granular Base (SSGBAH) crosswalk design.

Figure 112 FWD Testing on SSGBAH

The Load Transfer Efficiency (LTE) is a useful tool for determining how well the load is being carried across a joint or in this case the load that is being transferred from the asphalt section to the crosswalk section. The LTE is the ratio of unloaded deflection to the loaded deflection, which is given by;

LTE (%) = D_u/D_l*100 (Equation-4)

Where, LTE= Load Transfer Efficiency (percentage)

D_u= Unloaded deflection (mils)

D_l= Loaded Deflection (mils)

Table 13 below shows the Load transfer efficiency for each crosswalk.

Table 13 Results of FWD Testing

Drop ID	Test Type	D2 (+300 MM)	D9 (-300 mm)	LTE (%)	Location
4	Leave	16.83	12.18	72.4	SSGBAH
6	Leave	5.71	7.25	100.0	SSCBCH
7	Leave	18.34	14.69	80.1	SSABAH
8	Leave	5.99	7.75	100.0	BSCBCH

Figure 113 shows the deflection basin of each crosswalk. The BSCBCH and SSCBCH LTE is 100% which is maximum, it indicates that the load is being transferred across the joints. In the case of the SSGBAH and SSABAH crosswalks, the LTE is still above 70% which is an effective load transfer across the joints, indicating that they are performing in a good state at the present time. This analysis would also be consistent with the observed distress evaluations. In short, this analysis is complimentary to the distress analysis as it further indicates that the BSCBCH and SSCBCH are exhibiting the best performance at this point in time.

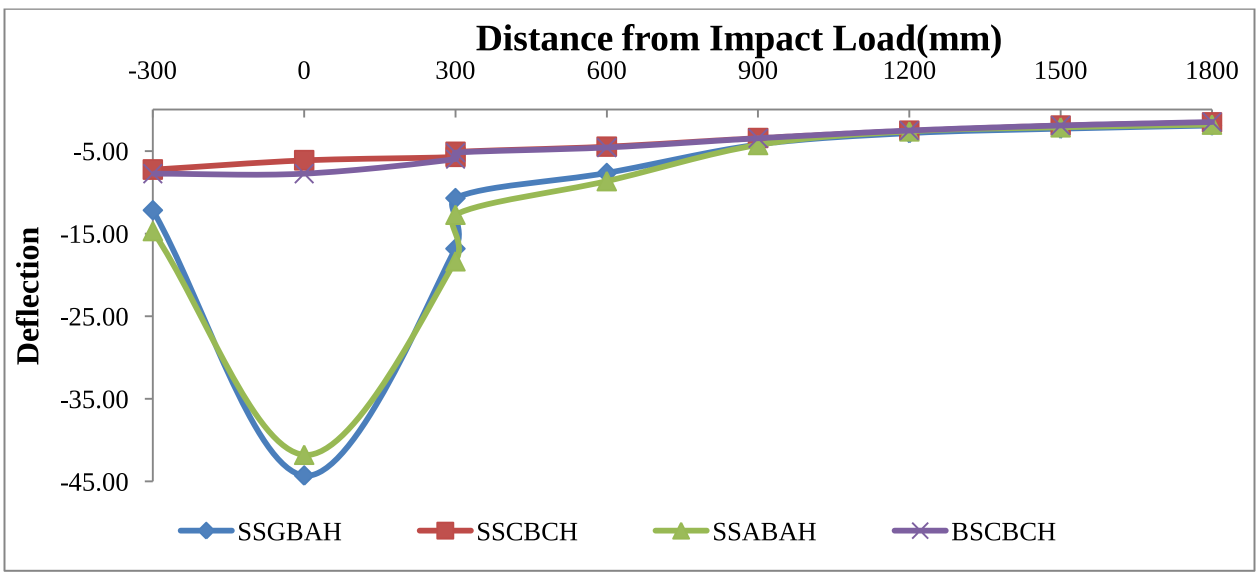


Figure 113 Deflection Basin of Four Crosswalks at Test Track

5.3 Pavement Distress Condition Evaluation

Distress condition surveys were carried out in accordance with the Interlocking Concrete Block Pavement Distress Guide developed by ICPI (ICPI 2008). The guide is based on the Pavement Condition Index (PCI) methodology and was modeled on the U.S. Army of Corps of Engineers MicroPAVER distress guide as published by ASTM. The PCI is a numerical indicator that evaluates the present condition of the pavement based on the surface distress. The type and severity of pavement is assessed by visual inspection of each crosswalk section surface condition of the pavement. The PCI does not measure the structural capacity nor does it provide direct measurement of skid resistance or roughness. The structural capacity of each section is evaluated by measuring strains, stresses, temperature and moisture at different locations in the pavement which is discussed in previous section.

The following distresses are measured and evaluated for each crosswalk section. The individual type of damage is rated separately with both degree and extent of the damaged being assessed. The degree of distress is rated high, medium, or low based on PCI numerical indicator. The PCI is calculated by using ICPI spreadsheet.

- Damaged Pavers
- Depressions
- Edge Restraint
- Excessive Joint Width
- Faulting
- Heave
- Horizontal Creep
- Joint Sand Loss/Pumping
- Missing Pavers
- Patching
- Rutting

The survey was carried out in March 2010. Not all the distresses are found in the crosswalks of this research project to date. Out of eleven distresses horizontal creep, missing pavers and patching are not identified. Faulting and Heaving were identified in the last progress report for SSCBCH at the Ring Road, but since this crosswalk was temporarily repaired, they were not present this time. The summary of the PCI of all the section including PCI rating, pavement age and ESAL according to the survey conducted in March 2010 are shown in the table below.

Table 14 Summary of Pavement Condition Evaluation in Test Track and Ring Road Sections

Location	Section	PCI	Type of Traffic	PCI Rating	Pavement age	Total ESAL	Rank
Test Track	SSCBCH	47	Dynamic	Fair	32 months	3,875,000	3
	SSABAH	59	Dynamic	Good	32 months	3,875,000	2
	BSCBCH	85	Dynamic	Excellent	32 months	3,875,000	1
	SSGBAH	46	Dynamic	Fair	7 months	875,000	4
Ring Road	SSCBCH	60	Accelerating turning	Good	32 months	290,000	4
	BSCBCH	73	Static	Very good	32 months	290,000	1
	SSABAH	67	Static	Good	32 months	290,000	3
	SSGBCH	71	Static, Accelerating turning	Very good	32 months	290,000	2

The BSCBCH at the Test Track is showing the best performance as compared to all the other crosswalks at the Test Track. This is also the case at the Ring Road. The PCI of SSCBCH at Ring Road has increased after the temporary repair by University of Waterloo. The PCI vs. time for the period of July 2008 to April 2010 is as shown in the Figures 114 and 115 below. It is expected that a minimum level of service would be at a PCI level of 40 as noted on both Figures. The ranking tabulated are also supported by other tests performed.

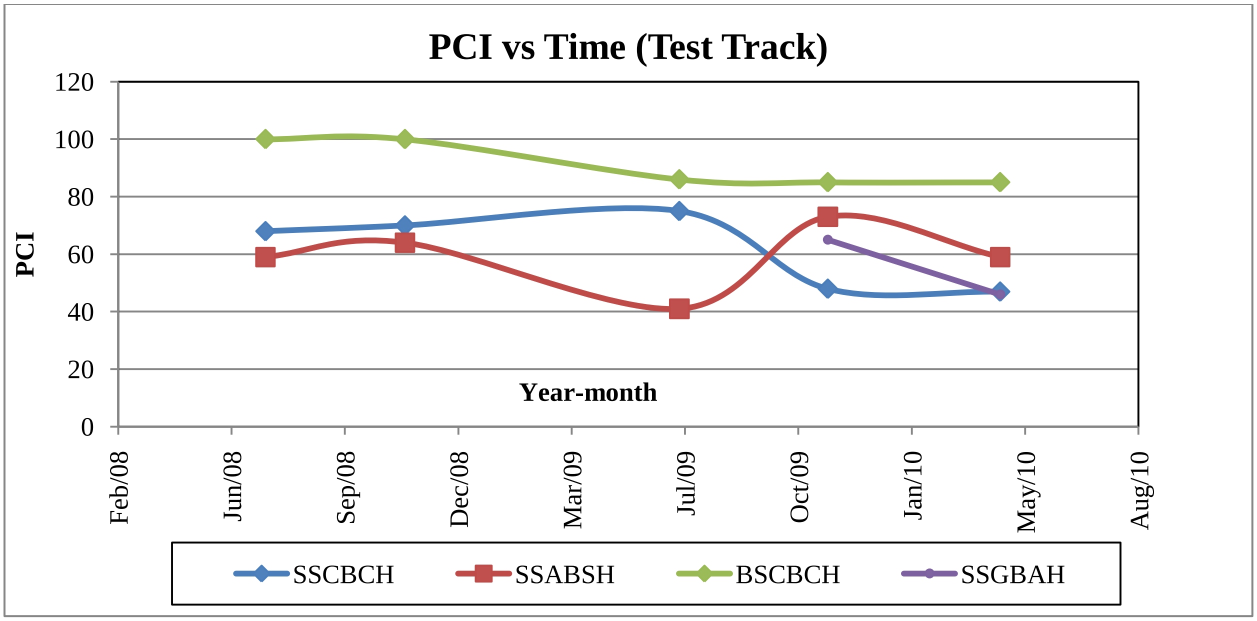


Figure 114 Pavement Condition Index of Test Track Over Time

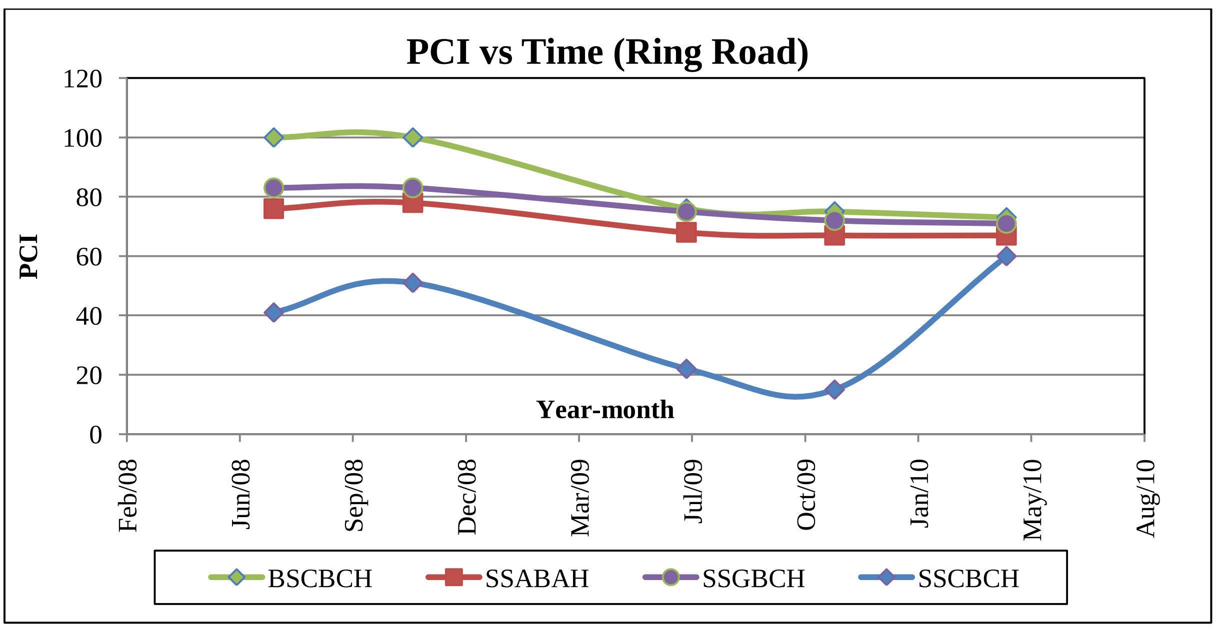


Figure 115 Pavement Condition Index of Ring Road Over Time

Comparing the PCI over time in both the locations, the visual distresses at the Test Track has increased significantly than in the Ring Road. This is due to the frequent movement of heavy construction equipment running at the Test Track in the summer of 2009, for the construction of new landfill site.

5.4 Profile

The CPATT SurPRO 2000 is a Class 1 multipurpose walking profiler manufactured by ICC as shown in Figure 116. The unit operates on a battery, two inertial inclinometers and an optical encoder. Data is collected by rolling the profiler at a constant speed along a line of interest. Inclinometers operate under the same principles as accelerometers; however, they are more sensitive to vibrations and are able to compute the angle of the profiler's inclination. Using the angle of inclination and distance travelled, the SurPRO 2000 calculates elevation changes with respect to the starting elevation.

The CPATT SurPRO was calibrated prior to starting the surveys. Longitudinal roughness profiles were taken in the left and right wheel paths and transverse profile were taken of each crosswalk as shown in the figure below. Prior to testing each crosswalk was inspected to ensure that they were free of any loose debris and profiling lines were marked with spray paint to help in reducing the meandering by operators. The test area is shown in the Figure 117.

Figure 116 SurPro 2000

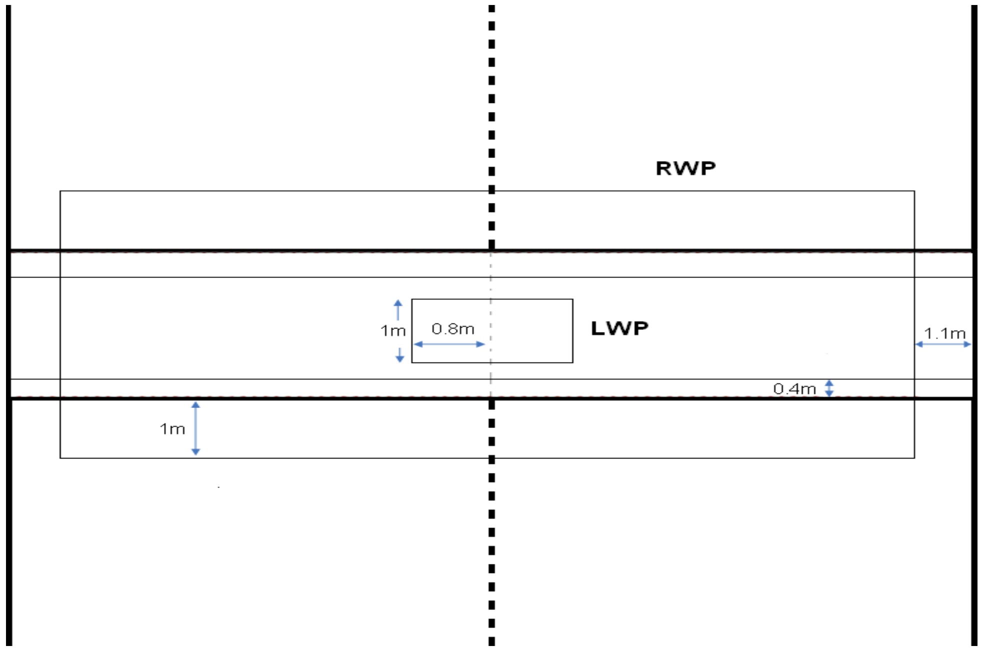


Figure 117 Test Section in the Crosswalk for SurPRO

***RWP: Right wheel path, *LWP: Left wheel path, * Transverse profile was taken 0.4m from the header of the crosswalk.**

The sampling interval of 20mm was used to get the profile. The data was collected at speeds below 3 km/h. The device was aligned with the starting line and slowly accelerated to avoid erroneous readings and gradually brought to a complete stop at a point beyond the end mark. Three runs were taken at each profile. The analysis of the data was done using ProVAL 2.7 software. The report of each profile is shown in the following sections. In the following profiles it should be noted that SurPRo operates in closed loop survey. The red marks are the start and end of the loop.

5.4.1 Test Track

Figure 118 to 121 shows the profile of north edge and south edge with respect to the centerline of the BSCBCH, SSABAH, SSCBCH and SSGBAH. The measured profile for each crosswalk is all parallel to each other in all the profiles, so it is very similar. The south lane which has loaded vehicles exhibits more rutting as compared to the North lane. However, there is one exception as the SSGBAH has more rutting in the north lane as opposed to the south lane. This is due to the rain after compaction of the granular base on the North lane.

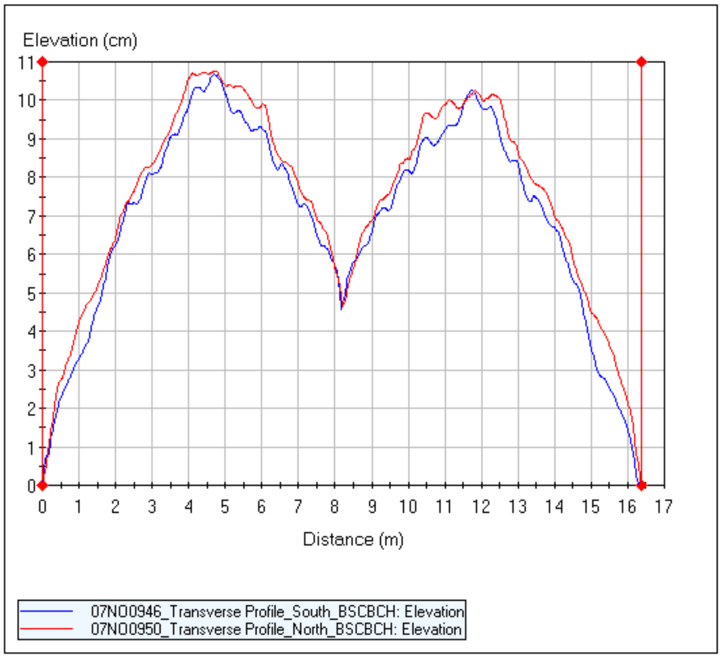


Figure 118 Transverse Profile of BSCBCH at Test Track

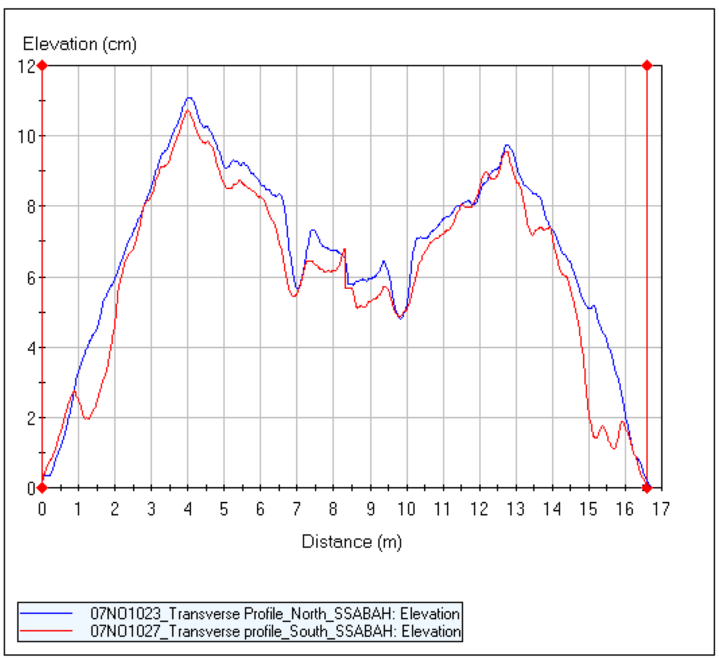


Figure 119 Transverse Profile of SSABAH at Test Track

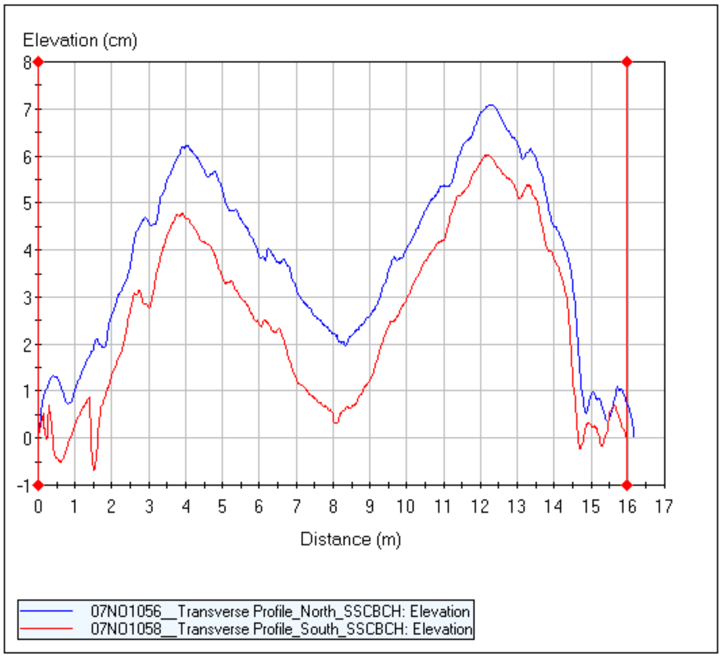


Figure 120 Transverse Profile of SSCBCH at Test Track

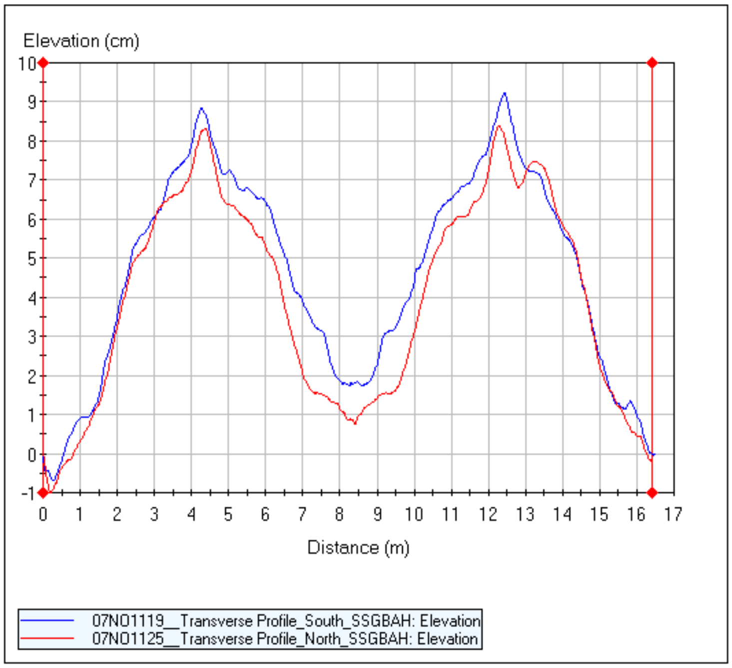


Figure 121 Transverse Profile of SSGBAH at Test Track

Figure 122 represents the longitudinal profile of the right wheel paths of all the crosswalks at the Test Track. It can be observed that there is more rutting in the SSCBCH crosswalk followed by the SSABAH crosswalk .This is also noted and consistent with the distress survey that are presented earlier in this progress report. The BSCBCH shows less rutting in comparison to the others.

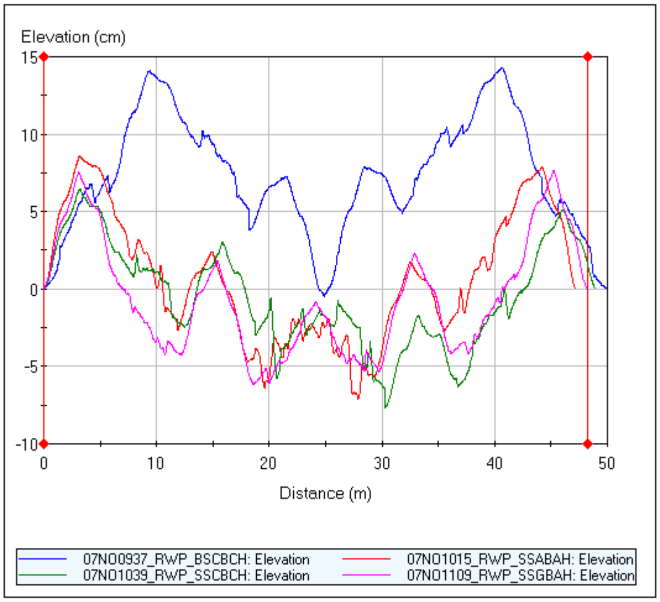


Figure 122 Longitudinal Profile at the Right Wheel Path of all Crosswalks at Test Track

Figure 123 represents the longitudinal profile of the left wheel paths of all the crosswalks at the Test Track. It can be observed that there is crack formation in the left wheel path of BSCBCH. High rutting is observed in the SSGBAH.

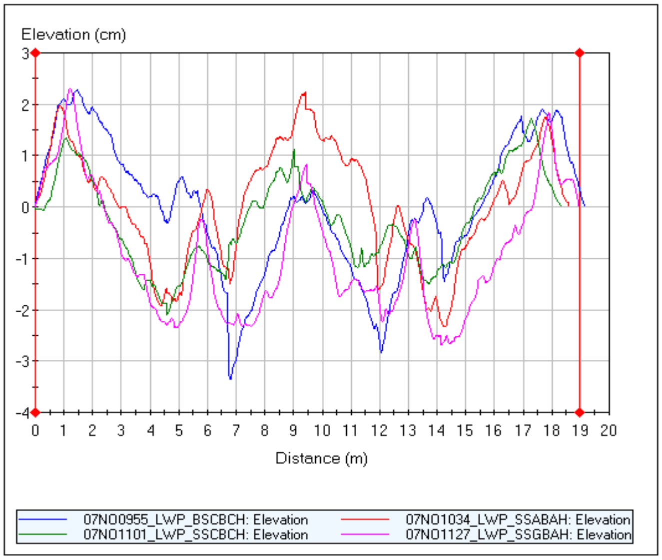


Figure 123 Longitudinal Profile at the Left Wheel Path of all Crosswalks at Test Track

5.4.2 Ring Road

Figure 124 to 127 shows the profile of the north edge and the south edge with respect to the centerline of the BSCBCH, SSABAH, SSCBCH and SSGBAH. In Figure 125 and 127, it is observed that more rutting is observed on the west side of the crosswalk as compared to the east side of the crosswalk. Similarly, in Figure 126, the south section has more rutting as compared to the north section.

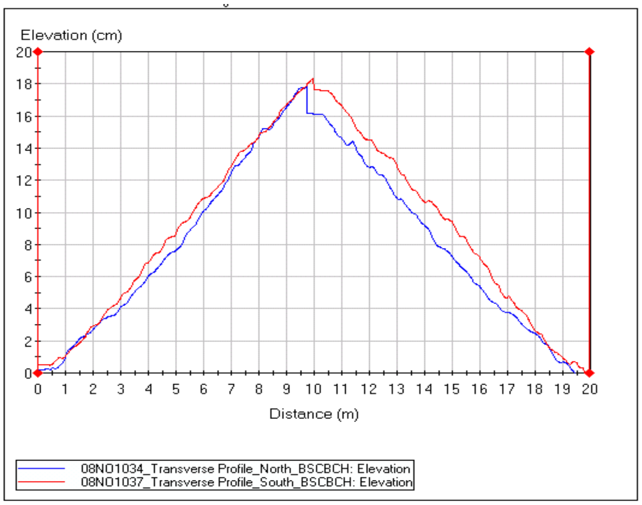


Figure 124Transverse Profile of BSCBCH at Ring Road

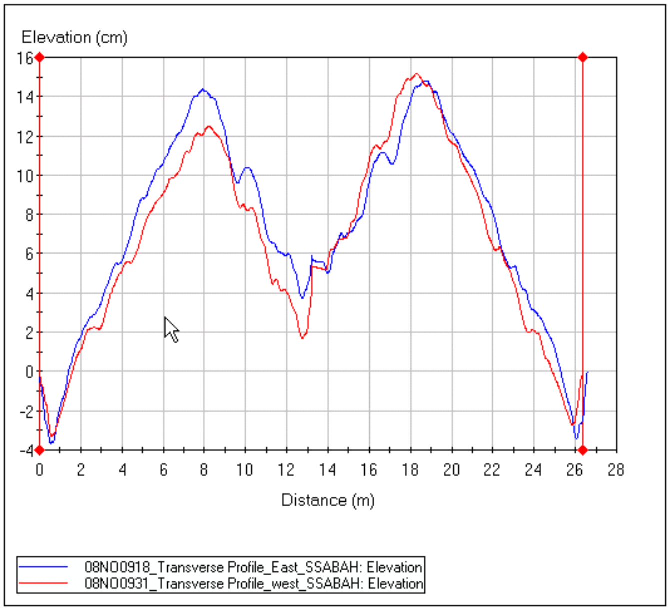


Figure 125 Transverse Profile of SSABAH at Ring Road

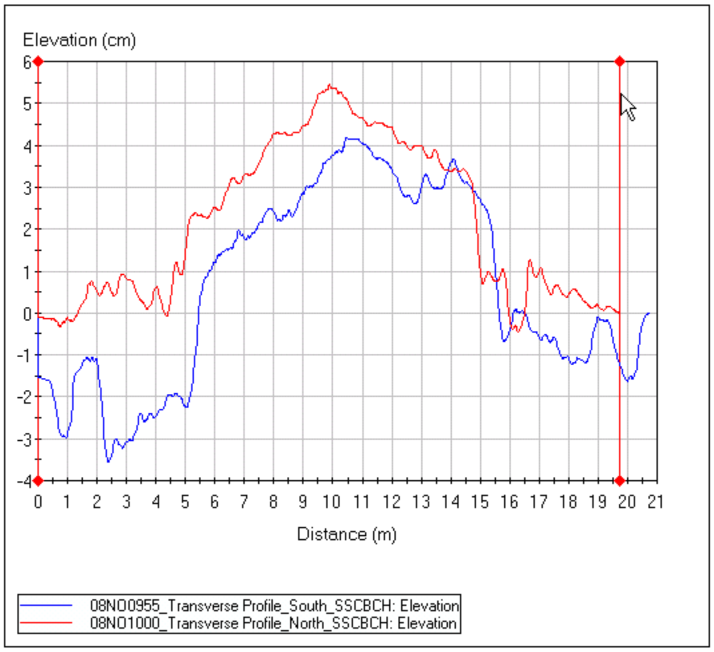


Figure 126 Transverse Profile of SSCBCH at Ring Road

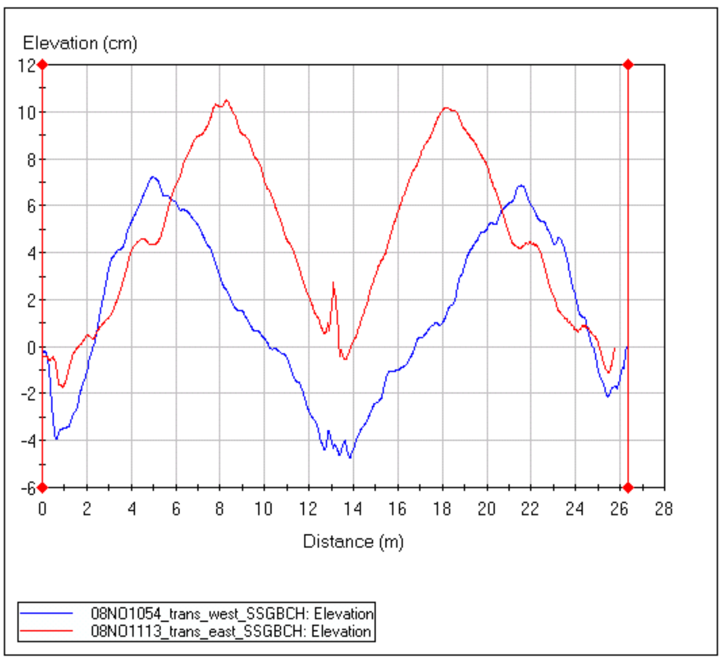


Figure 127 Transverse Profile of SSGBCH at Ring Road

Figure 128 represents the longitudinal profile of left wheel paths of all the crosswalks at the Test Track. It can be observed that there is more rutting in the SSABAH crosswalk followed by the SSGBCH crosswalk. The BSCBCH crosswalk shows less rutting in comparison to the others. The longitudinal profile of the SSCBCH was not taken due to maximum heaving, rutting and depressions along the path, which prevented carrying out SurPRO survey.

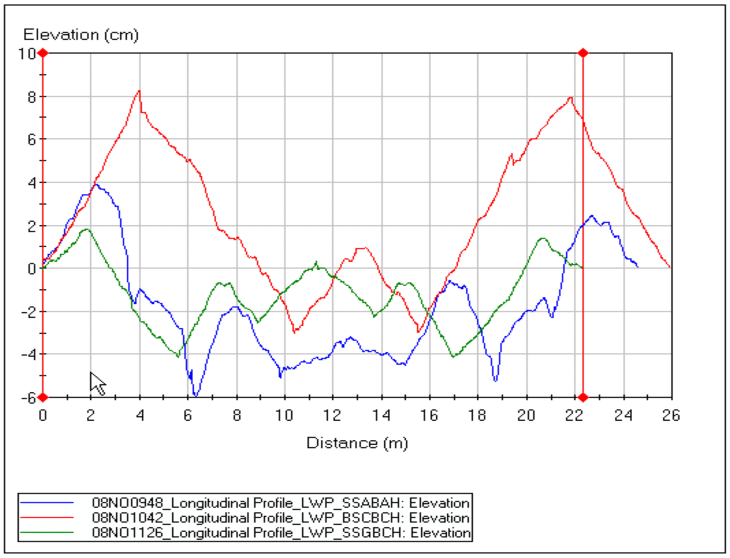


Figure 128 Longitudinal Profile at the Left Wheel Path of all Crosswalks at Ring Road

Figure 129 represents the longitudinal profile of the right wheel paths of all the crosswalks at the Ring Road. It can be observed that rutting is seen in the SSGBCH section followed by the BSCBCH and the SSABAH crosswalks in the right wheel paths.

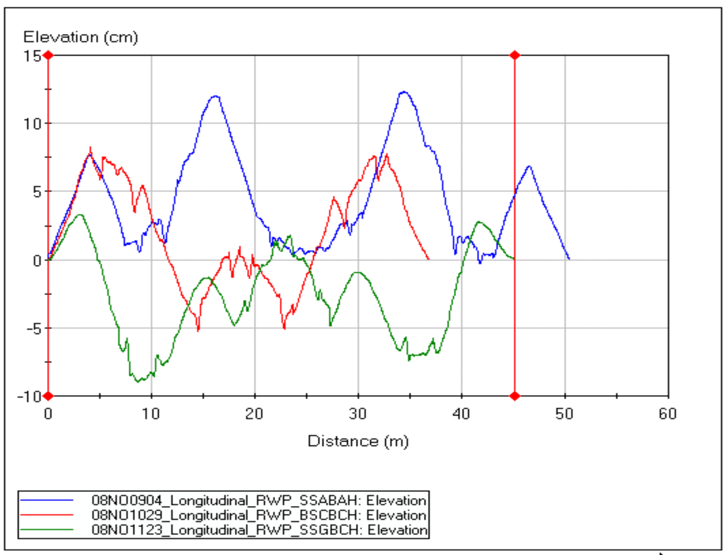


Figure 129 Longitudinal Profile at the Right Wheel Path of all Crosswalks at Ring Road

5.5 Summary

Table 15 Summary of Overall Observation at Test Track

Tests Conducted	**Location- Test Track**				
		SSABAH	**SSCBCH**	**BSCBCH**	**SSGBCH**
ESAL		3.75 million	3.75 million	3.75 million	875,000
Pavement Condition Index	Nov-09	73	48	85	65
	Apr-10	59	47	85	46
Deflection					
PFWD(average Deflection)		137 microns	286 microns	31 microns	459 microns
FWD	Load transfer Efficiency (%)	80.1%- In this crosswalk the LTE is still above 70% which is an effective load transfer across the joints.	100%-Deflection is high in this crosswalk yet the load transfer efficiency is excellent	100%- Deflection is very low in this crosswalk and load transfer efficiency is excellent.	72.4%-In this crosswalk the LTE is still above 70% which is an effective load transfer across the joints. Deflection is very high since it is a granular base crosswalk.
Profile		The Profile of the crosswalks is consistent across sections. The southbound lane has loaded vehicles and it exhibits more rutting as compared to northbound lanes. However, SSGBAH has more rutting in the northbound lane as opposed to southbound lane. This is likely due to the fact that during construction rain was encountered and this impacted the initial compaction and the overall quality of construction.			

Table 16 Summary of Overall Observation at Ring Road

Tests Conducted	Location- Ring Road				
		SSABAH	**SSCBCH**	**BSCBCH**	**SSGBCH**
ESAL		290,000	290,000	290,000	290,000
Pavement Condition Index	Nov-09	67	15	75	72
	Apr-10	67	60	73	71
Deflection PFWD (average Deflection)		162 microns	100 microns	26 microns	206 microns
Profile		The profile of the crosswalks shows that there is more rutting in SSABAH which is followed by SSGBCH. BSCBCH is the better performing crosswalk according to the profile information.			
Comments		Followed by SSCBCH, this crosswalk is good for City road condition, in the heavy loads condition like in Test Track, this crosswalk was repaired.	Followed by BSCBCH, this crosswalk was repaired and that is why it is performing better at this stage.	Low deflection i.e. more stiff, similar performance as of Test Track	Unlike Test Track, this crosswalk is performing better

Chapter 6

Overview, Observations and Recommendations

6.1 Overview

- Eight test sections were built at two sites in Waterloo, Ontario to assess the structural performance of four different interlocking concrete pavement crosswalk design assemblies under two different loading scenarios.
- Six sections were instrumented with mechanical and environmental sensors whereas aggregate base section on the Ring Road and Test Track has only environmental sensors.
- The pavement behaviour under 3,750,000 ESALs repetitions at the Test Track site and under 290,000 ESALs on the Ring road site was studied. The CPATT Test Track encounters heavy truck loading primarily loaded garbage trucks with maximum load up to 56,000 kg (6 axles) while the UW Ring Road traffic is similar to a typical urban road with approximately 10% truck and 5% bus traffic.

6.2 Observations

- At the Test Track, the stresses and strains are observed as follows: the maximum observed stress for all sections is measured in the asphalt base section (SSABSH) followed by bituminous set concrete base (BSCBCH) and sand set concrete base (SSCBCH) crosswalks. The maximum strain is observed in the bituminous set concrete base crosswalk (BSCBCH) followed by sand set concrete base (SSCBCH) and sand set asphalt base (SSABSH) crosswalks. Since the maximum strain and stress values are well below the allowable strain and stress from typical models, there is no indication at this time of unserviceable fatigue cracking and rutting in the sections.
- At the Ring Road, the strains are observed as follows: maximum observed tensile strain for all test sections is found in the SSABAH section followed by the BSCBCH and the SSCBCH crosswalks. The maximum tensile strain has not exceeded the maximum allowable strain. Therefore no unserviceable fatigue cracking has occurred in the sections in this period. It is observed that, the lowest performance is on the SSCBCH crosswalk which was due to inadequate surface drainage design which caused severe ponding of surface water runoff on this crosswalk in the spring. However, it has been repaired temporarily in November 2009.

- According to the deflection survey, the maximum deflection is observed in SSGBCH/SSGBAH at both the location. The SSCBCH crosswalk has the second highest deflection at the Test Track and third at the Ring Road. The SSABAH is second highest at the Test Track and third at the Ring Road. While the BSCBCH has the highest stiffness and lowest deflection in both the sections. All the deflections observed in the crosswalk are less than a half a mm. The deflections for the crosswalks at both Test Track and Ring Road are listed below.

Table 17 Deflection of the crosswalks

Crosswalk Assembly	Test Track Deflection (mm)	Ring Road Deflection (mm)
SSCBCH	0.232	0.013
BSCBCH	0.005	0.011
SSABSH/SSABAH	0.049	0.026
SSGBAH/SSGBCH	0.097	0.043

- According to PCI rating, the BSCBCH is performing the best at both sites. The PCI of BSCBCH is seen to be consistent after 85 for Test Track and 73 for Ring Road. The Ring Road assembly with only 300,000 ESALs has less PCI then the one at Test Track assembly with 3.5 million ESAL. This is due to different traffic loading and speed condition. At Test Track we have uniform speed of 70 km/hr and at Ring Road there is static traffic, slow moving and accelerating which resulted in causing more distress than in Test Track section.

  SSCBCH at the Ring Road and the SSABAH at the Test Track are rated good. It should be noted that the SSABAH was repaired in July, 2009. The maximum deformation, rutting and depression are observed in the SSCBCH section at the Ring Road site and in Test Track. The SSGBCH section at the Ring Road is performing the second best followed by the SSABAH and the SSCBCH sections.

Table 18 Ranking of the Crosswalks According to the Performance

Location	Crosswalks	PCI	Deflection		Rutting	Rank	Comments
			PFWD (microns)	FWD (LTE %)			
Test Track	BSCBCH	85	31	100	Minimal amount of rutting	1	
	SSCBCH	47	286	100	25mm to 30mm rutting in the wheel paths of southbound lane	3	
	SSABAH	59	137	80.1	20mm rutting in southbound lane	2	After repair
	SSGBAH	46	459	72.4	20mm rutting in all wheel paths	4	Pavement age is six months, very high rutting is seen which is the early sign of failure.
Ring Road	BSCBCH	73	26		Minimal amount of rutting	1	
	SSCBCH	60	100		Rutting is low but depression is 25mm	4	PCI increased after temporary repair
	SSABAH	67	162		20mm rutting in the bus park	3	
	SSGBCH	71	206		Minimal amount of rutting	2	

6.3 Recommendations

- Additional monitoring of pavement performance is needed to observe longer term trends.
- Based on review of the data and conservative engineering judgment, the recommended maximum lifetime 80 kN equivalent single axle loads are in Table 19 below. These are based on a PCI of 60 as the lowest acceptable condition and the basis for rehabilitation, as well consideration of deflections and load transfer:

Table 19 Maximum Recommended Lifetime Design ESALs for Various Crosswalk Assemblies

Crosswalk Assembly	ESALs/PCI						Maximum recommended lifetime design ESALs, millions
	Jun-09		Nov. 2009		Apr-10		
	Test track	Ring Road	Test track	Ring Road	Test track	Ring Road	
BSCBCH	240,000/100	19,000/100	2.4M/85	260,000/75	3.75M/100	290,000/73	7.5
SSCBCH	240,000/68	19,000/44	2.4M/48	260,000/15*	3.75M/47	290,000/60	2
SSABAH	240,000/61	19,000/75	2.4M/73**	260,000/67	3.75M/59	290,000/67	1.5
SSGBCH	No section	19,000/82	New section	260,000/72	875,000/46	290,000/71	0.7

*Repaired November 2009 **Rehabilitated July 2009

The crosswalk SSABSH at Test Track failed and was rehabilitated on July 2009. During this process Steel Header was replaced by Aluminum Header. This failure of crosswalk was recognized as construction deficiency so it was not considered in determining the lifetime design ESAL. It should be noted that there is heavy channelized loading in the Test Track in the case of SSGBCH.

In addition to all the comments, one of the important points to consider is, the quality of workmanship in the construction of the crosswalks for this research purpose was excellent.

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