A Pilot Study of Instrumented Unbonded Concrete Overlay in Toronto

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Abstract

Unbonded concrete overlays provide structural rehabilitation and can be cost effective when the existing pavements are highly distressed and/or removal of existing pavement layers is not desirable. This paper describes a pilot study, which involves a three way partnership between the University of Waterloo, the Cement Association of Canada and the City of Toronto, to evaluate and document the performance of the first instrumented unbonded concrete overlay in Canada. The rehabilitation area is located at a heavily-trafficked intersection on Bloor Street and Aukland Road in the City of Toronto. The large volumes of bus traffic from the nearby subway station caused significant damage to the pavement structure. The rehabilitation design was composed of placing an unbonded concrete overlay on Bloor Street and replacing the section on Aukland Road with full depth exposed concrete. Twelve strain gauges are embedded in the concrete layer and they are strategically located along the wheel paths of the buses. The sensors are being monitored to assess long-term performance of the concrete overlay and the full depth concrete subjected to traffic and climatic loads. This paper outlines the design parameters, site conditions, and material properties of the overlay. The construction and full-scale instrumentation of this project are described, followed by preliminary analyses of up-to-date performance records.

Introduction

The majority of roadway projects in North America over the next few decades will involve rehabilitation of existing pavement infrastructure (TAC 1997). Unbonded concrete overlay is one viable method that has been effectively used to rehabilitate concrete and composite pavements. This rehabilitation strategy is characterized by an

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asphalt concrete layer or bond-breaking layer of about 25mm (1 inch) between the Portland cement concrete (PCC) overlay and the in-service concrete pavement. The use of the interlayer minimizes the occurrence of reflection cracking transmitted from the existing pavement. PCC unbonded overlays perform equivalent to slabs that are supported by a firm subgrade (Hall). Unbonded concrete overlay becomes more attractive and cost effective when existing pavements are highly distressed and full depth removal is not desirable. Another advantage of the unbonded concrete overlay is that joints in the existing pavement do not have to be cleaned, sealed or matched with the joints of the overlay. There are no unique techniques required for the construction of unbonded overlays and preparation of the surface is not as critical as that of a bonded resurfacing (Hall). This University of Waterloo research project involves a pilot study, in collaboration with the Cement Association of Canada and the City of Toronto, to monitor and document the performance of the first instrumented unbonded concrete overlay in Canada. The study is conducted at a heavily-trafficked "T" intersection on Bloor Street and Aukland Road in the City of Toronto. Bloor Street is a four lane major arterial road extending Easterly and Westerly carrying about 30,000 vehicles per day with extremely heavy peak hour traffic. It is also a major bus route to the nearby Kipling Subway station through Aukland road as illustrated in Figure 1.

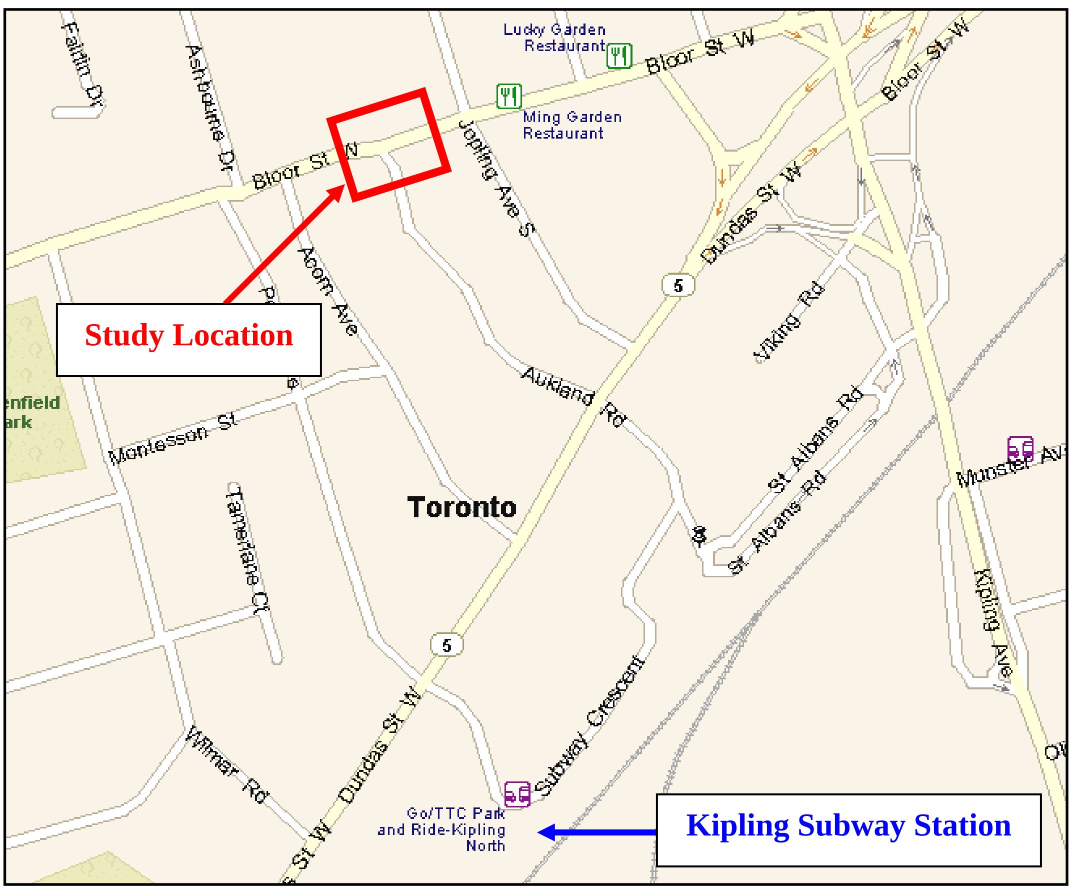

Figure 1 – Study Location: T-intersection on Bloor Street and Aukland Road

The portion of Bloor Street within the area of study was a composite pavement with 200 mm concrete base on 100-150 mm granular road base, overlaid by 75-80 mm asphalt concrete. The existing pavement structure on Aukland Road was composed of 190 mm asphalt concrete (150 mm HL8 and 40 mm HL1), over 250 mm granular road base.

This intersection was significantly deteriorated by the large volume of bus traffic from/to the Kipling Subway Station. High severity rutting with ruts up to 150 mm in depth was observed at the Bloor Street traffic signals. Rutting was particularly evident at stop bars in the curb lanes on Bloor Street to Aukland Road. In addition, the road had several distresses related to the numerous utility cuts. The rehabilitation design involved placing an unbonded concrete overlay on Bloor Street at Aukland Road and replacing a section of Aukland Road with full depth exposed concrete. The construction took place during the construction season in July/August of 2003. Twelve strain gauges are embedded in the concrete layer (seven on Bloor Street and five on Aukland Road) and they are strategically located along the wheel paths of the buses. The sensors are being monitored to assess the long-term performance of the concrete overlay and the full depth concrete subjected to the heavy bus traffic and the extreme climatic loads in Toronto.

Scope and Objectives

The overall goal of this pilot study is to increase pavement life and serviceability through the development of rehabilitation strategies that are both technically and economically effective. Validation of strategies through systematic observation is a critical element in this process.

This paper outlines the design parameters, site conditions, and material properties of the overlay. The construction and full-scale instrumentation of this project are described, followed by results from the preliminary analyses of up-to-date performance records.

Structural Evaluation: Pre and Post Construction

Trow Associates Inc., a geotechnical consultant was retained by the City of Toronto to undertake the structural evaluation of the intersection. Falling Weight Deflectometer (FWD) tests were performed in early June 2003 to obtain pre-construction data. The same exercise was repeated directly prior to construction, at the beginning of July to assess the existing concrete slab on Bloor Street and the granular base on Aukland Road. In early September, a post rehabilitation assessment was performed to determine the final pavement structural condition. The increase in structural capacity was estimated by determining the in-situ resilient moduli of the various layers using a commercial back-calculation program, ELMOD.

The testing program divided the intersection into eight locations. It was designed to cover the three lanes on Aukland Road including one southbound lane (SBL), one northbound driving lane (NBD), and one northbound turning lane (NBT) and Bloor Street which consisted of five lanes, the eastbound passing lane (EBP), eastbound driving lane (EBD), westbound passing lane (WBP), westbound driving lane (WBD), and centre-line median lane (CLM). A summary of the FWD back-calculated moduli and statistical analysis for the eight pavement locations before, during, and after construction are presented in Tables 1, 2 and 3. Figure 2 shows a schematic of the lanes at the intersection.

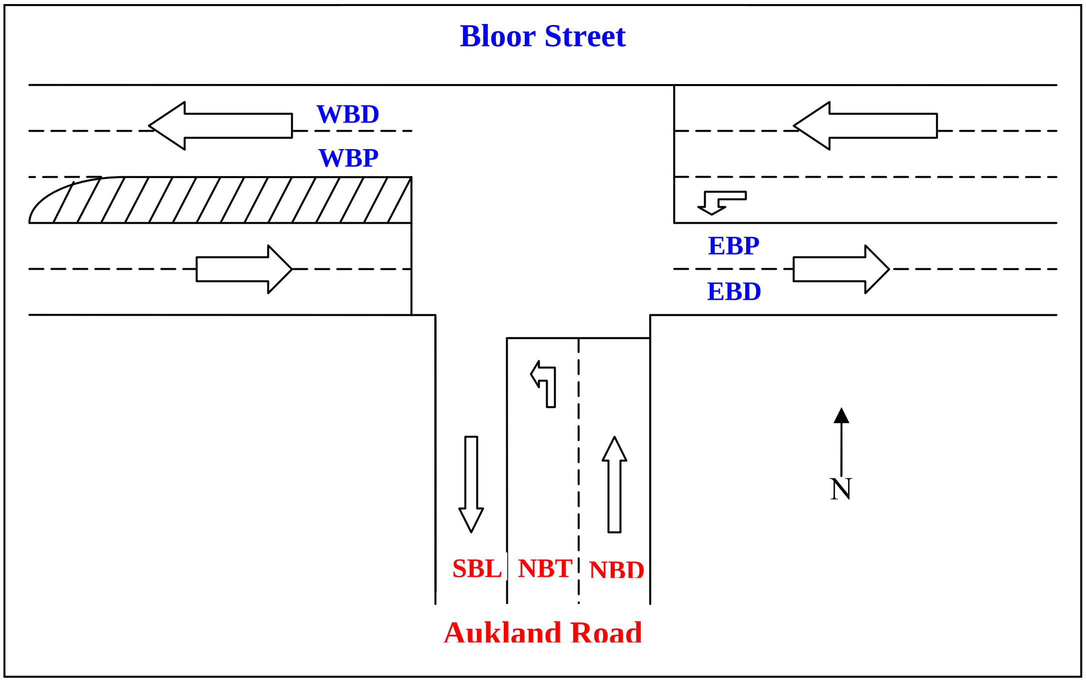

Figure 2 – Schematic of the Lanes at the T-intersection

Table 1-Pre-construction Back-calculated Layer Resilient Moduli (Trow 2003)

a. Aukland Road

Location	Thickness		Back-calculated Resilient Modulus		
	HMA (mm)	Granular (mm)	HMA (MPa)	Granular (MPa)	Subgrade (MPa)
SBL	190	260	2086 (182)	195 (12)	47 (6)
NBT	190	260	2063 (144)	198 (7)	49 (5)
NBD	190	260	2064 (182)	196 (12)	45 (7)

b. Bloor Street

Location	Thickness			Back-calculated Resilient Modulus			
	HMA	Concrete	Granular	HMA	Concrete	Granular	Subgrade
	(mm)	(mm)	(mm)	(MPa)	(MPa)	(MPa)	(MPa)
EBD	80	200	150	2156 (201)	22108 (1310)	200 (8)	46 (5)
EBP	80	200	150	2139 (139)	22607 (1282)	199 (11)	43 (7)
CLM	80	200	150	2090 (168)	22971 (1680)	203 (13)	44 (6)
WBP	80	200	150	2206 (215)	21665 (1597)	199 (10)	42 (6)
WBD	80	200	150	2179 (174)	22775 (1698)	206 (9)	45 (7)

1. The back-calculated moduli are mean values from 5 to 9 samples
2. The values in brackets in the moduli columns are standard deviations

Table 2 - During Re-construction Back-calculated Layer Resilient Moduli (Trow 2003)

a. Aukland Road (no HMA layer)

Location	Thickness	Back-calculated Resilient Modulus	
	Granular	Granular	Subgrade
	(mm)	(MPa)	(MPa)
SBL	260	204 (12)	49 (8)
NBT	260	197 (17)	46 (6)
NBD	260	195 (14)	50 (3)

b. Bloor Street (no HMA layer)

Location	Thickness		Back-calculated Resilient Modulus[1]		
	Concrete	Granular	Concrete	Granular	Subgrade
	(mm)	(mm)	(MPa)	(MPa)	(MPa)
EBD	200	150	22825 (1828)	211 (8)	47 (6)
EBP	200	150	22861 (1445)	192 (11)	45 (7)
CLM	200	150	22719 (1439)	194 (14)	47 (3)
WBP	200	150	22230 (1477)	205 (16)	44 (6)
WBD	200	150	22172 (1535)	195 (15)	46 (7)

1. The back-calculated moduli are mean values from 5 to 9 samples
2. The values in brackets in the moduli columns are standard deviations

Table 3 - After construction Back-calculated Layer Resilient Moduli (Trow 2003)

a. Aukland Road

Location	Thickness		Back-calculated Resilient Modulus[1]		
	Concrete (mm)	Granular (mm)	Concrete (MPa)	Granular (MPa)	Subgrade (MPa)
SBL	225	260	30684 (1448)	239 (19)	50 (3)
NBT	225	260	29720 (1791)	235 (16)	42 (8)
NBD	225	260	29176 (1678)	236 (18)	54 (5)

b. Bloor Street

	Thickness				Back-calculated Resilient Modulus[1]				
Location	Concrete (mm)	HMA (mm)	Concrete (mm)	Granular (mm)	Concrete (MPa)	HMA (MPa)	Concrete (MPa)	Granular (MPa)	Subgrade (MPa)
EBD	150	80	200	150	29918 (1419)	3023 (128)	20347 (1161)	207 (13)	51 (6)
EBP	150	80	200	150	29531 (851)	3007 (72)	20153 (1257)	210 (11)	46 (9)
CLM	150	80	200	150	30345 (946)	2975 (102)	19787 (1150)	212 (13)	49 (11)
WBP	150	80	200	150	30442 (1223)	2936 (116)	20108 (1162)	216 (9)	48 (8)
WBD	150	80	200	150	28805 (1400)	3086 (121)	20308 (1928)	194 (2)	53 (3)

1. The back-calculated moduli are mean values from 5 to 9 samples
2. The values in brackets in the moduli columns are standard deviations

Interpretation of the back-calculation data led to the conclusions that the old asphalt (HMA) layer on Bloor Street and Aukland Road was in moderate to severe condition structurally. The deterioration and severe rutting was the main reason for the replacement of the wearing surface. The existing in-service concrete slab on Bloor Street showed moderate level of strengths. During the rehabilitation, isolated repairs (rout and seal) were performed on the slab before the concrete overlay was placed. In addition, some areas required grinding. The replacement of the fresh base granular materials on Aukland Road subsequently increased the resilient modulus of the layer. The HL3 debonding layer (which is a typical municipal asphalt mix in Ontario) on Bloor Street had a high strength at the time of post-reconstruction testing. The new concrete surface layer at the intersection had very high in place strength, according to the back-calculated results. The coring results confirmed that the depth of existing layers ranged from 60mm-100mm of asphalt concrete over 202 to 215mm of PCC with a granular base of approximately 150mm-200mm over a silty clay subgrade.

New Pavement Design

Since a concrete wearing surface can better accommodate the stopping, turning, and accelerating of the buses, as compared to an asphalt concrete surface, it was decided that the area 30 m east and 30 m west of the "T" intersection along Bloor Street at Aukland Road would receive an unbonded concrete overlay. Figure 3 provides a schematic of the two concrete overlay designs. The existing asphalt concrete surface was completely removed by milling to the underlying concrete base. The condition of the concrete base was thoroughly checked to identify areas requiring repair (rout and seal). The concrete base was then tacked lightly with SS-1 emulsified asphalt before placing a 25 mm high stability HL3 asphaltic concrete layer as the debonding layer, followed by placing a 150 mm lift of 32 MPa concrete. The transition at both ends of the test strip extended the overlay to the total length of 85 m. Grid pattern contraction joints (1.5 m maximum spacing) were cut to a depth of L/4 (L being the thickness of the concrete) using a "wet" saw cut method, as soon as concrete was set (4 to 12 hours). Dowel bars were spaced at 1.4 m intervals prior to placing the 150 mm concrete overlay on the 25 mm HL3 debonding layer along major turning locations and stopping areas for buses. The dowel bars are 25 mm in diameter and 400 mm long, and they were placed on special chairs designed to locate the bars at the middle of the third section of the concrete slab. Table 4 outlines the concrete mix design.

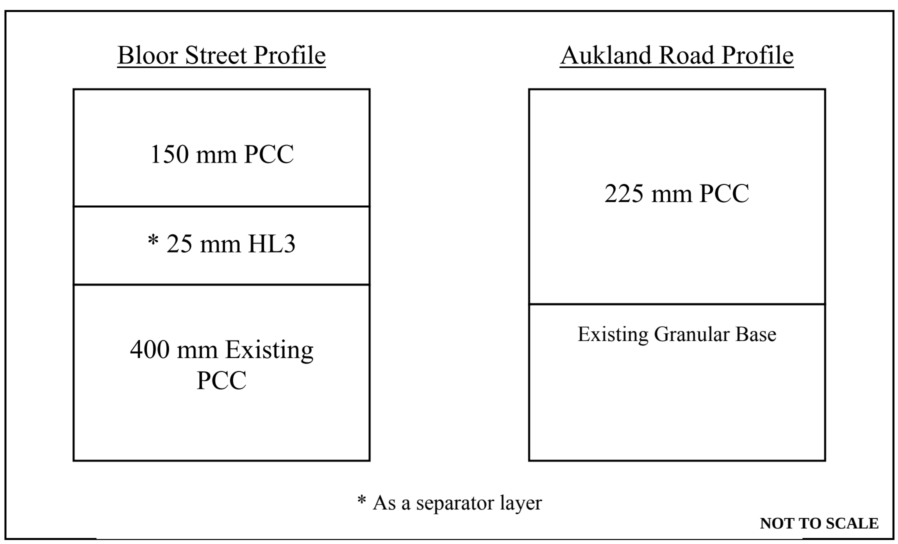

Figure 3 - Concrete Overlay Designs for Bloor Street and Aukland Road

Table 4- Concrete Mix Design (Tighe 2003)

Minimum Compressive Strength (MPa)	32
Accelerated Strength (MPa)	20 in 72 hrs.
CSA Exposure Class	C2
Maximum Water/Cement Ratio	0.45
Minimum Cementitious Content (kg/m^3)	335
Cement Type	10
Nominal Size Coarse Aggregate (mm)	19
Stone Type	Limestone
Plastic Air Range (%)	5-8
Slump Range (mm)	80 maximum
Yield	1.001

The new structural design for Aukland Road resulted in removing the existing 190 mm of asphalt concrete and replacing it with 225 mm full depth 32 MPa concrete pavement 63 m southerly from Bloor Street. Before the full depth concrete was placed, approximately 150 mm of the granular base course layer was replaced with fresh material and was re-graded to the design elevation. The concrete pavement at Bloor Street was separated by an expansion joint. Contraction joints were located in a grid pattern 4.5 m apart from each other. Dowel bars (25 mm in diameter, 400 mm long) were hand-placed on special chairs set at the middle third section of the concrete slab, within the first two contraction joints at the north and south ends of the rehabilitated area.

Because this intersection serves as the gateway to the nearby Kipling Subway Station, the high traffic volumes required that traffic be disrupted as little as possible and therefore strict staging was implemented during the construction period. A closure was co-ordinated with the public transit authorities and weekend work was scheduled to complete the construction. The complete staging process is illustrated in Figure 4. The contractor was allowed to shut traffic down to one lane in each direction while work was carried out as noted in Stages 1, 2 and 3. The sensor installation took place during Stages 2 and 3. The cosntruction took place during two weekends.

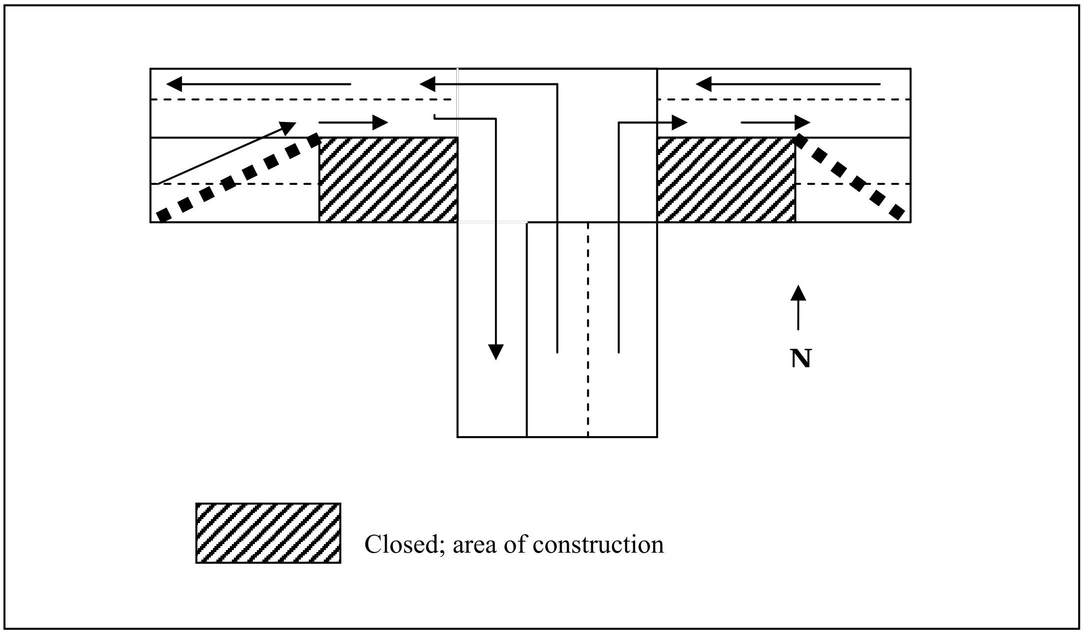

Figure 4a – Stage 1 of Construction

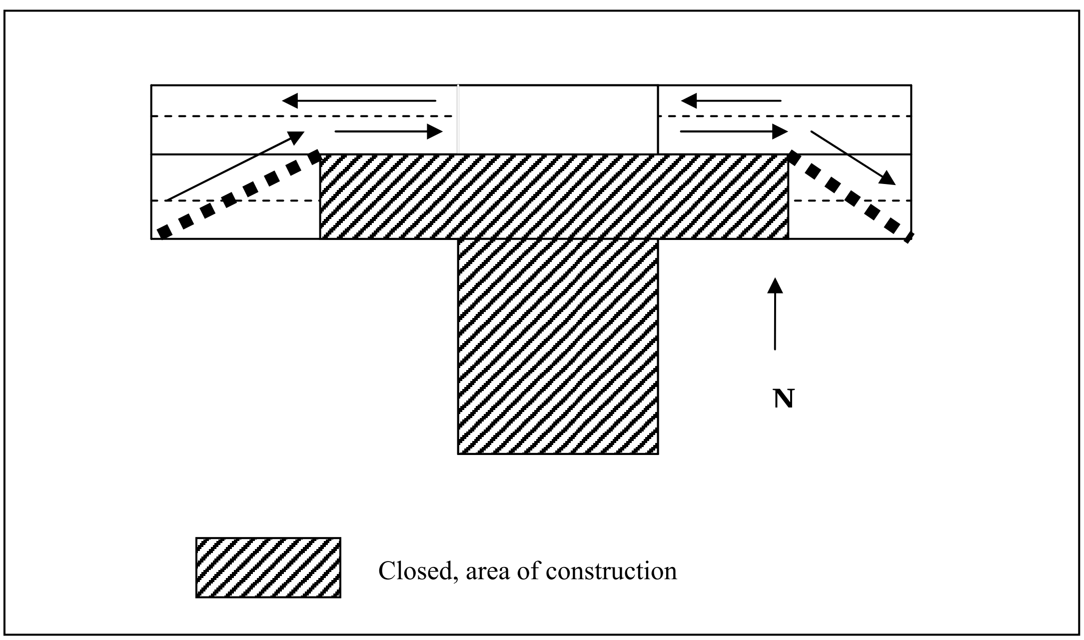

Figure 4b – Stage 2 of Construction

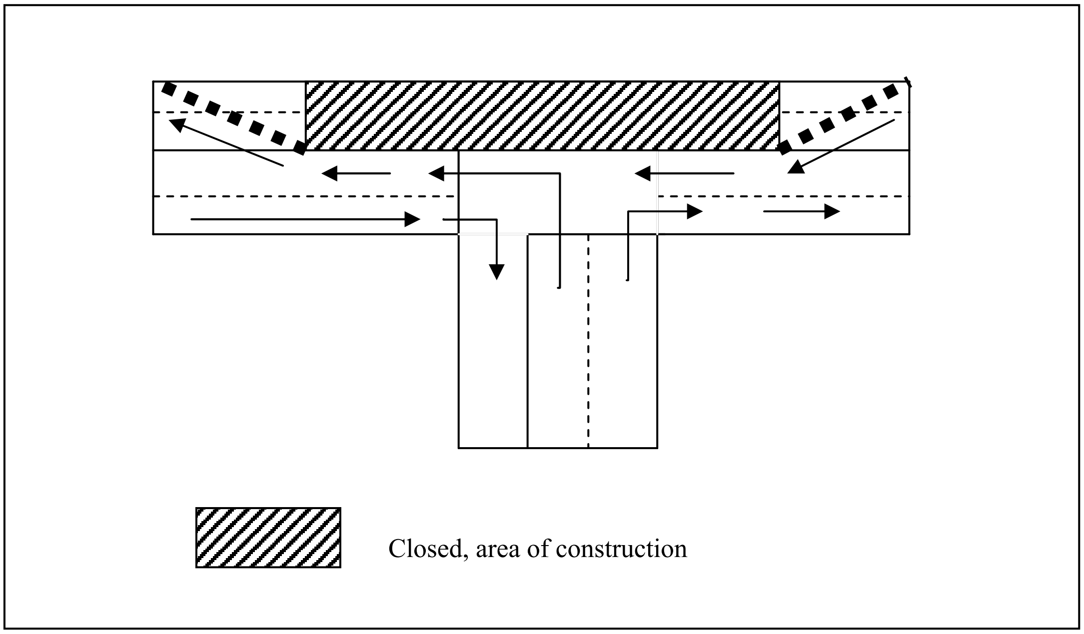

Figure 4c – Stage 3 of Construction

Sensor Installation and Monitoring

There is limited information in the literature with respect to the behaviour and performance of PCC overlays for pavements. Even though a number of States in the United States (for example, Route I-44 in Missouri) have implemented concrete overlays over existing flexible and rigid pavements, limited data is available. It is believed that the monitoring that is being carried out in this pilot study shall provide valuable performance information on these designs.

Sensors that were installed to monitor the performance of the unbonded concrete overlay on Bloor Street and full depth exposed concrete on Aukland Roads were the vibrating wire embedment strain gauges (model no. 52640199). The strain gauge is designed to measure the change of strain and the change of temperature in the concrete. Figure 5 shows the steel tube with flanges at both ends.

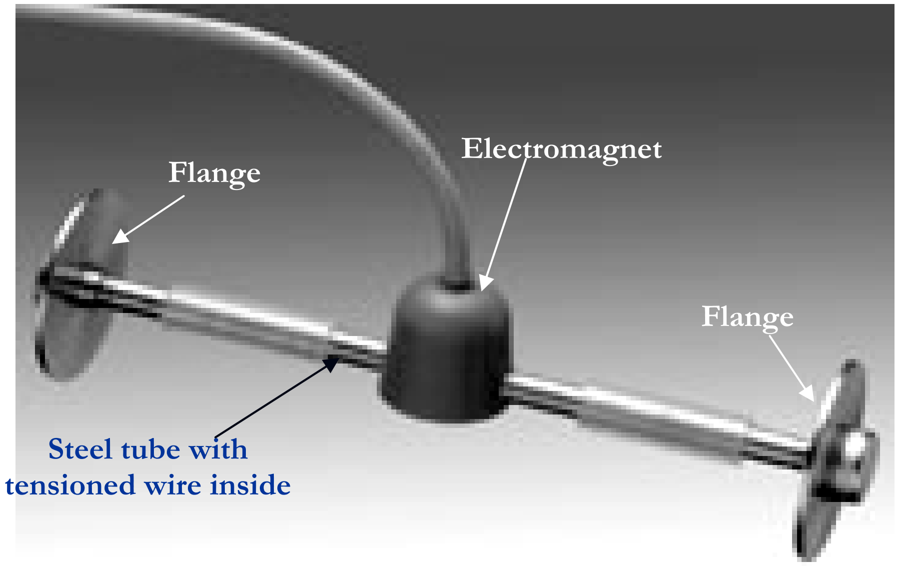

Figure 5- Vibrating Wire Embedment Strain Gauge

The vibrating wire embedment strain gauges operate on the principle that a tensioned wire, when plucked, vibrates at its resonant frequency. The square of this frequency is proportional to the tension in the wire (Slope 2003). Since the strain gauge is embedded in concrete, when the concrete volume changes, the two end flanges will move and as a result shorten or elongate the tensioned wire. An electromagnet coil is attached to the gauge body, which is used to pluck the wire and emits a signal to the data recorder of the resulting resonance frequency. Different frequencies are generated from different lengths of the wire; therefore, the corresponding change of strain ($\Delta\mu\varepsilon_M$) can be calculated by finding the difference between the initial resonance frequency reading and subsequent reading, and then multiplying it by a gauge factor (Slope 2003).

$$\Delta\mu\varepsilon_M = (R_1 - R_0)\times(GF) \qquad \text{[Equation 1]}$$

where R_1 and R_0 are the current and initial readings, respectively. In this case, the initial reading represents the first reading recorded after strain gauge installation. GF is the gauge factor which is related to the original length of gauge. It can be calculated as follows (Slope 2003):

$$GF = 0.1\times L^2 \qquad \text{[Equation 2]}$$

where L is the length of the wire in inches. In this project, the wire length is approximately 5.75” or 146 mm.

The strain gauge can directly measure actual strain changes due to changes in moisture content of the concrete and stresses from traffic loading. However, the change in strain due to temperature changes has to be adjusted by a thermal correction factor ($\Delta\mu\varepsilon_T$). This correction factor is required as the steel wire inside the gauge expands and contracts at a slightly different rate than the concrete during the temperature change. The thermal correction factor can be simply calculated by the multiplication of the differences between the thermal coefficient of expansion of concrete and steel and the change of temperature as follows (Slope 2003):

$$\Delta\mu\varepsilon_T = (TC_C - TC_S) \times (T_1 - T_0) \qquad \text{[Equation 3]}$$

where TC_C and TC_S are the thermal coefficient of expansion for concrete and steel wire, respectively. T_1 and T_o are the current and initial temperature readings, respectively. In this case, the values of TC_C and TC_S are 10 ppm/°C and 12 ppm/°C, and T_1 is the first temperature reading recorded after strain gauge installation (Slope 2003). The actual change of strain in concrete ($\Delta\mu\varepsilon$) due to temperature and traffic loading can be obtained as the difference between measured change of strain and the concrete thermal strain correction factor (Slope 2003).

$$\Delta\mu\varepsilon = \Delta\mu\varepsilon_M - \Delta\mu\varepsilon_T \qquad \text{[Equation 4]}$$

A negative value indicates compressive strain and a positive value indicates tensile strain in all of the above equations. This tensioned wire is held between the two end flanges of an embedment strain gauge. The loading of the concrete slab changes the distance between the two flanges. This results in a change in the tension of the wire. An electromagnet inside a strain gauge is used to pluck the wire and measure the frequency of the vibration. Strain is then back-calculated by applying calibration factors to the frequency measurement. Besides strain, these gauges can also measure temperature.

Twelve strain gauges were installed at strategic locations at the "T" intersection where seven of them were placed on Bloor Street and the rest on Aukland Road. The layout of the sensors is shown on Figure 6. Table 5 provides details on the location depth and wheel path location of the various sensors. The purpose of the variation is to monitor the pavement strain with respect to traffic. Sensors 4 to 7, 11, and 12 were placed directly underneath the bus wheel paths at locations where transit buses stop for passengers and traffic signals. Sensors 8 to 10 should capture the effects of buses turning on to Aukland Road from Westbound Bloor Street while Sensors 1 to 3 will receive accelerating traffic on Westbound Bloor Street.

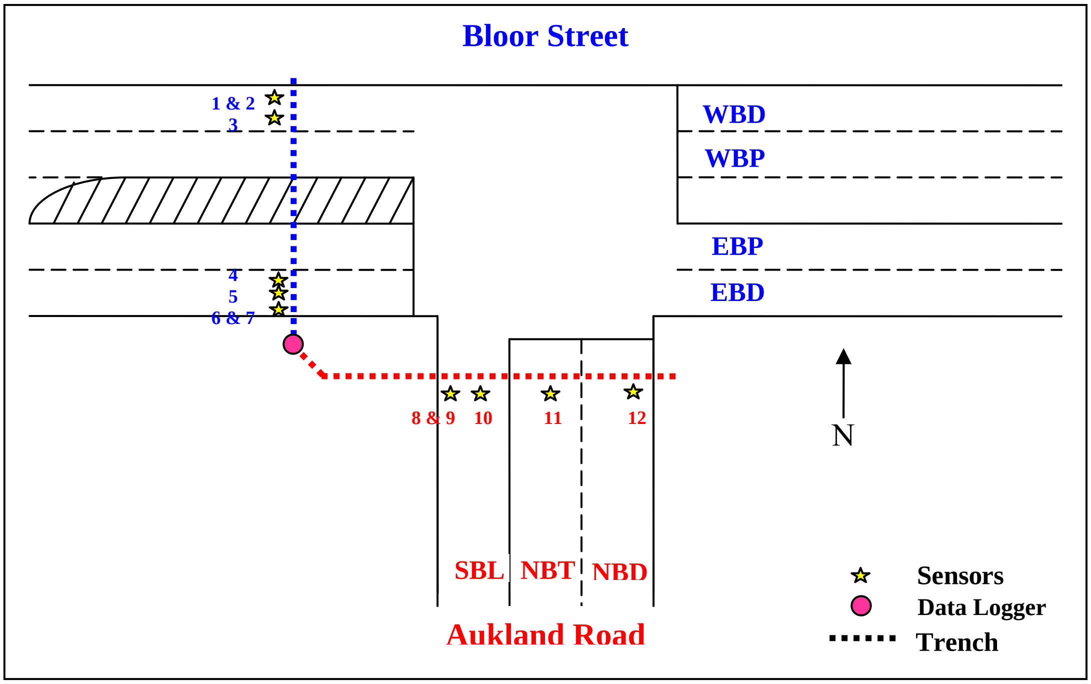

Figure 6 – Locations of the Sensors

Table 5 – Sensors' Locations and Purposes

Sensor	Street Name	Lane	Traffic Direction and Movement Type	Depth (mm) (Below PCC Surface)	Underneath Left or Right Wheel Path
1	Bloor Street	1	WBD	50	Right
2		1	WBD	150	Right
3		1	WBD	50	Left
4		2	EBP	50	Right
5		3	EBD	50	Left
6		3	EBD	50	Right
7		3	EBD	150	Right
8	Aukland Road	4	SBD	50	Right
9		4	SBD	225	Right
10		4	SBD	50	Left
11		5	NBT	50	Right
12		6	NBD	50	Right

WBD – Westbound Driving Lane
EBP – Eastbound Passing Lane
EBD – Eastbound Driving Lane
SBD – Southbound Driving Lane
NBD – Northbound Driving Lane
NBT – Northbound Turning Lane

Out of the twelve strain gauges installed, nine of them were placed at the bottom of the concrete slabs while Sensors 2, 7, and 9 were set on special chairs that brought the three sensors to 50 mm below the concrete surface, as illustrated in Figures 7 and 8. The gauges are placed in the orientation that the two flanges are in the same direction as the traffic. The intention of this arrangement was to capture any curling and warping stresses of the concrete slabs.

Figure 7 – Chairs for Sensor Installation 50 and 150mm below surface

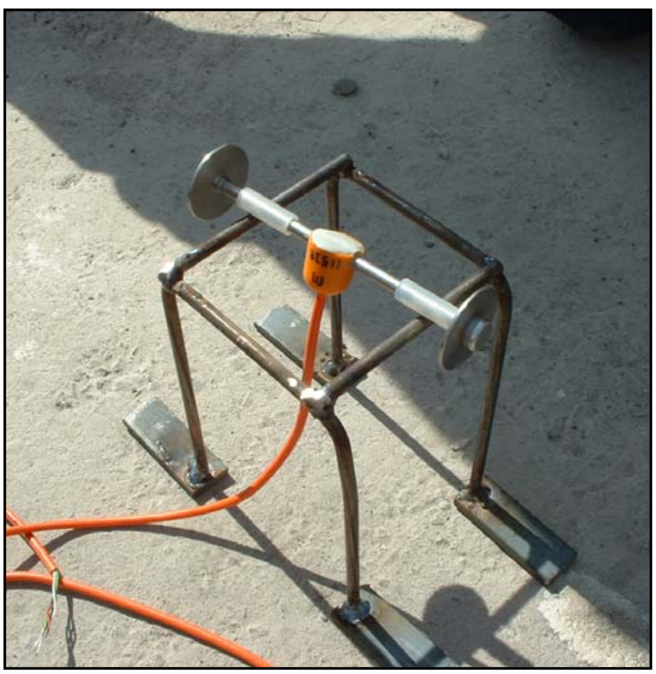

Figure 8 – Sensor and Chair

PCC Volume Changes Mechanism

The actual change of strain in concrete, measured by the strain gauges, is the total strain induced by the changes in concrete moisture content, changes in temperature, and changes in stress from traffic loading. The following section will discuss the general concrete volume change mechanism (strain) due to the traffic, moisture content, and the temperature.

The amount of strain changes in the PCC pavement is affected by the concrete boundary condition and the unbonded medium. There are two boundary conditions: restrained and un-restrained, which are depicted in Figure 9.

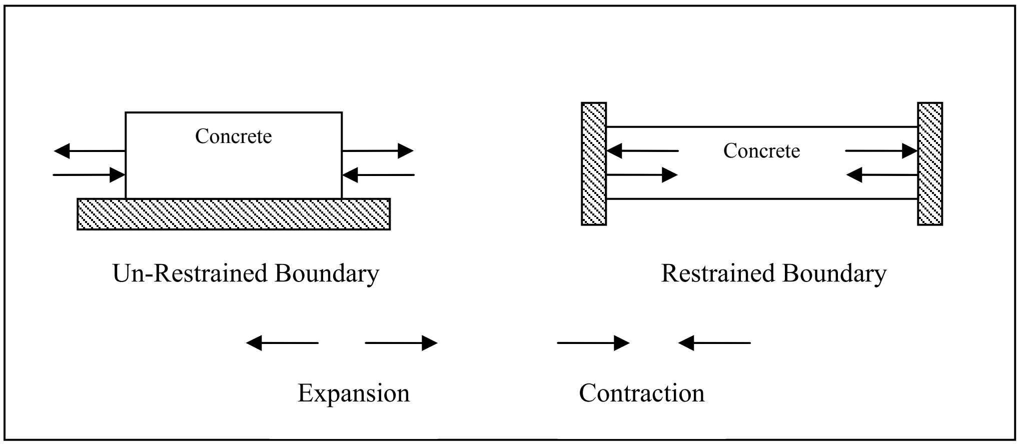

Figure 9 - Un-Restrained and Restrained Conditions

It is assumed only horizontal movement is allowed and the concrete movement can be measured by the strain gauge. The un-restrained boundary condition occurs when the concrete is on a smooth surface and can expand/elongate or contract/shorten freely without restriction. The concrete will not undertake any tensile or compressive stresses, but strain will occur in this case (Beer 1992). Alternatively, the restrained boundary condition occurs when the concrete is restricted from moving by two fixed end supports, which induce tensile or compressive stresses in concrete without strain occurring.

During contraction the PCC pavement acts under an un-restrained boundary condition. However, PCC pavement exhibits a combination of un-restrained and restrained boundary conditions when expanding. This is because PCC overlay pavement allows for expansion, but the expansion is limited. After the allowed volume of expansion has occurred, the road edge acts as a restraining boundary. Furthermore, although the unbonded medium was levelled before the PCC overlay was placed, it cannot produce a frictionless surface for the PCC overlay due to its aggregate content. Therefore, PCC pavement experiences a mixture of restrained and un-restrained boundary conditions.

Normally, the induced compressive stresses in concrete that a restrained boundary condition creates are not a concern as concrete has a high ability to withstand compression. However, the tensile stresses in concrete can lead to cracking

(ACI 209 2003). Furthermore, concrete volume changes associated with moisture content, temperature, or applied stress can result in changes. The following section provides a general concrete behaviour under all conditions.

Changes in Moisture Content

Changes in moisture content in concrete can cause concrete shrinkage and creep. Concrete shrinkage is the decrease in volume of concrete over time after hardening. This decrease is due to changes in the moisture content of the concrete without any external stresses (ACI 209 2003). However, the concrete's volume will not change at the beginning of drying as first water from the capillary pores, which is known as "free water", is removed. The removal of free water does not directly cause shrinkage. Shrinkage can be measured under steady conditions of relative humidity and temperature. Concrete shrinkage can cause concrete surface cracking. Experimental result shows that there are two type of drying shrinkage: irreversible moisture movement and reversible moisture movement.

When the concrete shrinkage reaches its ultimate compressive strain and is then stored in a saturated air condition for an extended period of time, the concrete will not recover to its original strain. The strain loss is known as irreversible shrinkage and will become a permanent deformation. The strain recovered from the ultimate compressive strain is known as reversible shrinkage (Nemati 2004).

Inversely, creep is defined as an increase in strain in hardened concrete that has been subjected to sustained stresses over an extended period. There are two kinds of concrete creep: basic creep and drying creep. Basic creep occurs without the moisture content in concrete changing, and drying creep occurs during a reduction in the moisture content (ACI 209 2003).

There are some other factors that affect drying shrinkage and creep other than relative humidity in the surrounding environment and the amount of loading. Table 6 summarizes the other factors which will affect the drying shrinkage and creep strain.

Table 6-Effect of Drying Shrinkage and Creep from Other Factors

Other Factors	**Symbol**
Loading Age (Creep Only)	γ_{la}
Ambient Relative Humidity	γ_{λ}
Average Member Thickness	γ_{h}
Slump	γ_{s}
Fine Aggregate Percentage	γ_{ψ}
Cement Content	γ_{c}
Air Content	γ_{α}

The ultimate shrinkage strain (ε_u) and creep coefficient (v_u) can be determined by the Equations 5 and 6 (ACI 209 2003). The ultimate creep coefficient defined as the ratio of creep strain to initial strain (ACI 209 2003).

$$\varepsilon_u = 780\gamma_{sh} \times 10^{-6} \qquad \text{[Equation 5]}$$

$$\upsilon_u = 2.35\gamma_c \qquad \text{[Equation 6]}$$

where γ_{sh} and γ_c are the production of all the correction factors which are listed in Table 6. Each correction factor has its own mathematical formula for shrinkage and creep. Details can be found from ACI Manual 209R "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures" (ACI 209 1992). According to Table 6, most of the correction factors of the creep and shrinkage strains are the concrete composite properties and geometries related. Concrete composite properties and geometries are the constant variables in the concrete pavement. Therefore, the change of creep and shrinkage strains vary due to loading age, applied stresses, and change in moisture content in concrete.

Temperature is the second major environmental factor in creep and shrinkage. Temperature changes on concrete creep and shrinkage can affect the time ratio rate and the rate of concrete aging (ACI 209 2003). At 50°C, creep strain is approximately two to three times as great as the temperature between 19°C to 24°C. At temperatures of 50°C to 100°C, creep strain will continue to increase with temperature and can reach four to six times more than the strain that is experienced at room temperature (ACI 209 2003).

In this study, the environmental condition (ambient humidity) and the applied load (traffic load) vary with respect to time. Due to the fact the pavement is only one and a half years old at this point in time, it has been assumed that any developed shrinkage or/and creep strains are considered as reversible and are not significant in this study. This is further emphasized by the visual condition of the pavement to date.

Temperature Changes

Volume change in concrete is also affected by temperature changes. When the temperature increases, concrete tends to expand as shown in Figure 10. However, the volume change differentials and restraint condition result in tensile strains and stress that may cause cracking.

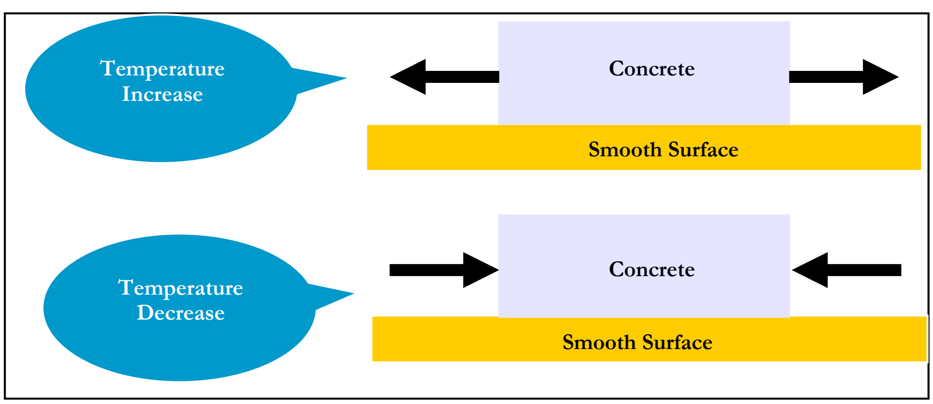

Figure 10 - Concrete Behaviour in Temperature Changes

Concrete has a low thermal conductivity and heat escapes from a body inversely as the square of its least dimension (ACI 207 2003). However, the loss of heat from hardened concrete is much faster than the loss of moisture from hardened concrete. The behaviour of exposed surfaces of concrete is greatly affected by daily and annual cycles of ambient temperature. It means the temperature of the concrete pavement surface is completely responding from the variation of the daily air temperature (ACI 207 2003). Only 10% of daily variation in air temperature can affect the concrete temperature at a depth of 0.6 m from the surface (ACI 207 2003). The annual ambient temperature affects the concrete at much greater depth. Only 10% of annual variation in temperature can affect the concrete temperature at a depth of 7.5 m from the surface (ACI 207 2003). Due to the different degree of ambient temperature effect on concrete pavement depth, it shows that the surface is subjected to a more severe tensile strains and stresses caused by temperature changes. Temperature strain is known as thermal-induced strain and it can be calculated by using a simple formula under an un-restrained condition:

$$\mu\varepsilon_T = (TC_C)\times(T_1 - T_0) \qquad \text{[Equation 5]}$$

where TC_C is the thermal coefficient of expansion for concrete which is same as Equation 3. For ordinary thermal stress calculations, when the type of aggregate in concrete and the concrete degree of saturation is unknown, thermal coefficient of expansion for concrete of $10\text{x}10^{-6}$ /°C may be sufficient (ACI 209 2003). This value is the same as the value we use for strain gauge temperature correction. T_1 and T_o are the current and initial temperature readings, respectively.

Data Collection and Analysis Methodology

There are two ways of retrieving data. The current method involves using a direct wire which sends data from the data logger using the LoggerNet software to a lap-top computer. Logger Net is software that writes and compiles monitoring programs, to transfer the program to the data logger, and to retrieve data by direct wires or telephone modems when used with a CR 10X data logger. The data logger is powered by the solar panels shown in Figure 11. The other option is to retrieve data using a telephone modem. A telephone line is connected to a Com 200 telephone modem which is equipped with a data logger. A computer in Waterloo uses LoggerNet software to initiate communication and receive the data. While the computer is connected it is possible to have real-time data transfers.

Figure 11 - Solar Panels

The purpose of the strain gauges is to measure the change of volume (strain) of concrete in a horizontal direction. According to the literature review, the change of volume is due to the change of moisture content (creep and shrinkage) and change of temperature in concrete. Creep and shrinkage strains typically develop over a long period of time once the PCC pavement has been in service for one year. Therefore, the concrete shrinkage and creep strains are not noticeable in this preliminary study. Thus, for the purpose of this assessment it is assumed that temperature change is the most significant factor affecting the total change of strain in concrete. Therefore, in this preliminary study, all measured strain and measured concrete temperature data in each sensor will be plotted in a common time-scale to evaluate the relationship between temperature changes and the strain changes with respect to time. The comparison of the strain changes in different locations, such as the top of the layer, bottom of the layer, the various lanes, left wheel-path and right wheel-path will also be discussed.

Strain data was collected on an hourly basis from December 2003 to September 2004. For the most part, data collection has been continuous however, some technical difficulties were encountered. The available data period are listed on Table 7.

Table 7 - Data Collection Period (Leung 2004)

	Date		ROW		Time Scale	
Period	**From**	**To**	**From**	**To**	**From**	**To**
1	21-Dec-03	20-Jan-04	13	733	37976	38600
2	27-Jan-04	26-Feb-04	734	1254	38013	38043
3	22-Apr-04	22-May-04	1455	2175	38099	38129
4	06-Aug-04	05-Sep-04	2176	2896	38205	38235
5	06-Sep-04	21-Oct-04	2897	3534	38236	38263
6	08-Oct-04	07-Nov-04	3535	4255	38268	28398
7	30-Dec-04	29-Jan-05	4256	4976	38351	38381

Also, based on a preliminary analysis, some of the sensors provided erroneous data. Thus for this analysis, only the sensors listed in Table 8 will be used. Figure 12 shows the measured strain versus time of each functional sensor during the seven available periods mentioned above. The change in strain always compares the current value to the base value measured. This removes the base strain associated with the sensor. The concrete temperature is the measured in-situ temperature on the gauges.

Table 8 - Functional Sensors (Leung 2004)

Sensor Number	Street Name	Lane Number	Traffic Direction and Movement Type	Depth (mm) (Below PCC Surface)	Underneath Left or Right Wheel Path
1	Bloor Street	1	WBD	50	Right
2		1	WBD	150	Right
3		1	WBD	50	Left
6		3	EBD	50	Right
7		3	EBD	150	Right
8	Aukland Road	4	SBD	50	Right
10*		4	SBD	50	Left
* Only Temperature Data is available					

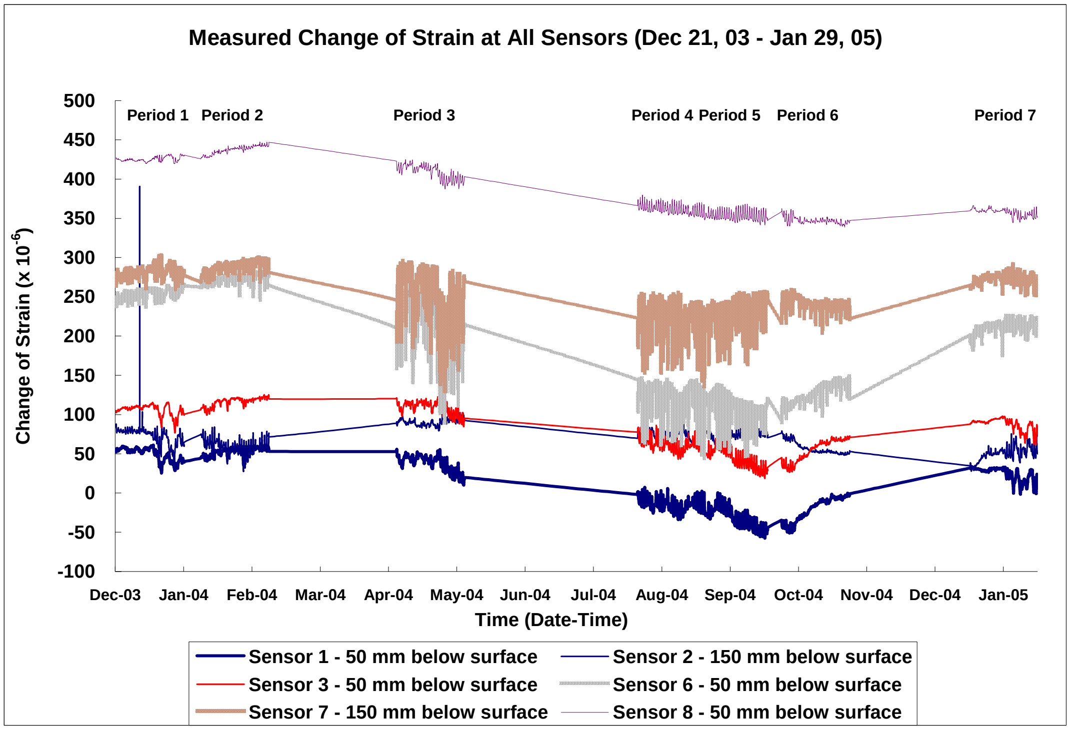

Figure 12 - Measured Strain versus Time Periods for All Sensors (Leung 2004)

As shown in Figure 12, the trends of temperature changes in concrete for all functional strain gauges are similar. In general, the overall change of strains over time is decreasing, which means the concrete compressive strains are increasing. An example of the change in strain and temperature versus time for each strain gauge in each period is shown in Figure 12. Again, this trend was consistent for all strain gauges over the various periods. The strain measurements from Sensors 1, 2, 3, and 8 all show that as the temperature increases, the strain also increases. This shows that temperature change is a significant factor in the driving/curb lanes. However, with sensors 6 and 7 which are located in the right wheel path where heavy vehicles are stopped (note this would include heavily loaded buses entering subway station) the measured strain does not increase with a temperature increase. It appears temperature change is not a significant factor at this location. This behaviour may possibly be related to the fact that the constant loading associated with the buses results in a larger impact on the strain as opposed to the temperature changes. Table 9 summarizes the sensor comparisons described herein.

Table 9 - Pavement Behaviour Comparison in Different Sensors (Leung 2004)

Sensors	Location	Comparison Behaviour
1, 2	50 mm and 150 mm below surface at driving lane (vehicles accelerating at this location)	Upper and Lower
6, 7	50 mm and 150 mm below surface at driving lane(vehicles stopped at this location)	Upper and Lower
1, 3	50 mm below surface at accelerating lane, left and right wheel-paths	Left and Right
1, 6, 8	50 mm below surface at accelerating, stopping, and turning lanes	Different Lane Movement
2, 7	150 mm below surface at accelerating and stopping lanes	Different Lane Movement

Sensors 1 and 2 are located at the right wheel-path way of driving lane on Bloor Street. Traffic at this location is primarily accelerating. The strain gauges are located at 50mm and 150mm depths below the surface. From the analysis the strain change at the lower depth (150mm) is higher than the strain change observed at 50mm (Leung 2004).

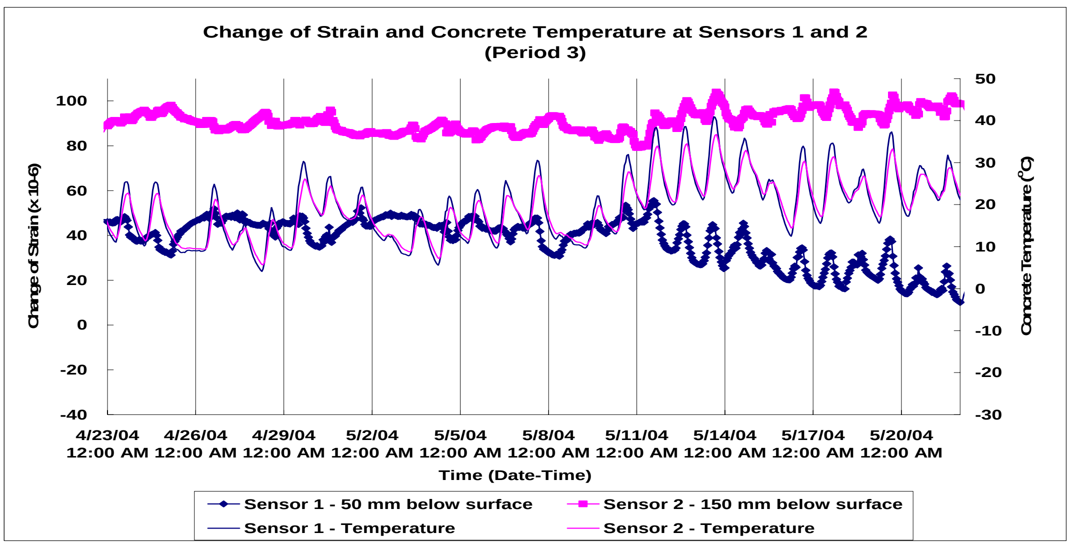

Figure 13 – Sensors 1 and 2 in Period 3 (Leung 2004)

Figure 13 shows the change in strain and temperature for Sensors 1 and 2 during April and May 2004. It is interesting to note that the strain observed with Sensor 1 located 50mm below the surface is quite different then the strain observed at Sensor 2 at 150mm below the surface. At this location most of this change is due to temperature changes and not static loading. Figure 14 shows the same time period for Sensors 6 and 7 where there is heavy static loading. As noted, the effect of the temperature is limited as the change in strain observed at 50mm and 150mm below the depth are similar and follow a similar range as compared to Sensor 1 and 2.

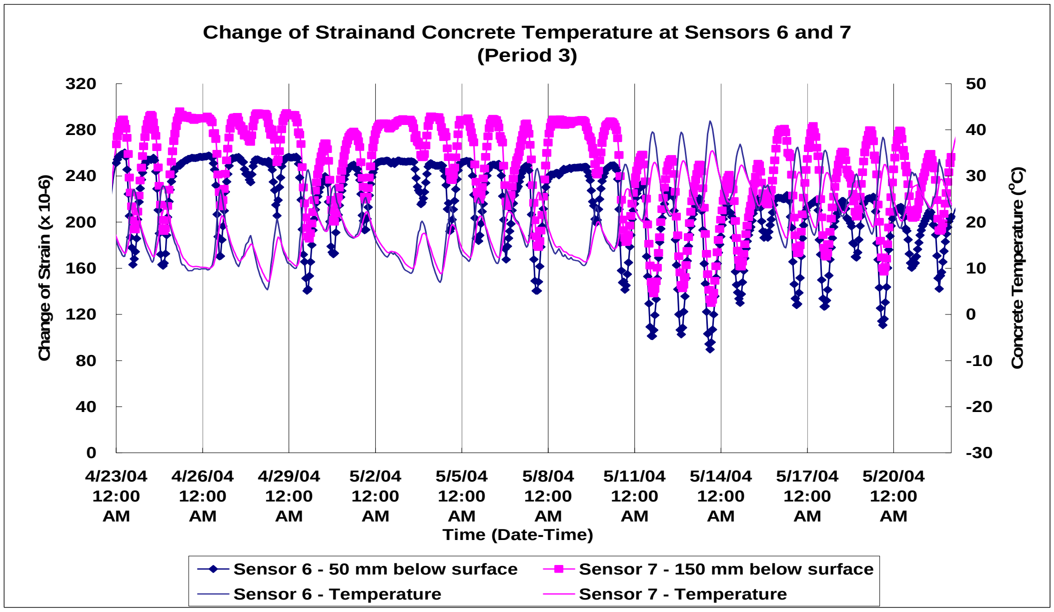

Figure 14 – Sensors 6 and 7 in Period 3 (Leung 2004)

The strain analysis for different lanes (Sensors 1, 6 and 8) shows that the PCC pavement has similar strain changes behaviour in accelerating and turning lanes in all periods. However, the change of strain at the stopping lane has a reverse response with the temperature changes. Also during Period 3 and Period 4, the strain changes undergo extensive fluctuations in the stopping lane. In the overall performance, the change of strain in PCC pavement decreased which would indicate the concrete is in a compressed state when the concrete temperature goes from negative air temperature to positive air temperatures.

Sensors 2 and 7 are both located 150 mm below the surface in the driving lane, but are located in different lanes (accelerating vs. stopping lanes). The change of strain at the stopping lane indicates higher compression levels as opposed to the accelerating lane throughout all four periods.

Statistical Analysis

Although the data to date is preliminary, a statistical analysis has been carried out to examine temperature and strain differences in the various sensors. The data has been grouped into seven periods as described earlier. As more data becomes available a more thorough statistical analysis is planned. Analysis of Variance (ANOVA): Two-Factor without Replication, was used to determine the significant difference in the measured strain and temperature of concrete pavement between two different strain gauges (sensors). The sensors were compared in pairs, which are listed in Table 10.

Table 10 - List of Sensor Comparison

Sensors	Location	Comparison Behaviour
1, 2	50 mm and 150 mm below surface at accelerating lane	Upper and Lower
1, 3	50 mm below surface at accelerating lane, left and right wheel-paths	Left and Right
1, 6	50 mm below surface at accelerating and stopping	Different Lane Movement
1, 8	50 mm below surface at accelerating and turning lanes	Different Lane Movement
2, 7	150 mm below surface at accelerating and stopping lanes	Different Lane Movement
6, 7	50 mm and 150 mm below surface at stopping lane	Left and Right
6, 8	50 mm below surface at stopping, and turning lanes	Different Lane Movement

Each sensor can measure concrete pavement temperature and change of strain; therefore, two separate ANOVA were performed in this report:

1. ANOVA for the measured temperature between sensors
2. ANOVA for the measured change of strain between sensors

The "ANOVA: Two-Factor without Replication" can test whether there is a significant difference between columns and between rows by a F-Test. In this case, columns are the measured strain or measured temperature in the compared sensors; rows are the time of measurement. Therefore, there are four null hypotheses in this analysis.

1. $H_{o(temp|time)}$: There is no significant difference in temperature between time
2. $H_{o(temp|sensor)}$: There is no significant difference in temperature between sensors
3. $H_{o(strain|time)}$: There is no significant difference in strain between time
4. $H_{o(strain|sensor)}$: There is no significant difference in strain between sensors

Null hypothesis will be rejected if F_{value} calculated from Excel ANOVA is greater than the $F_{critical}$ (i.e. There is significant difference between factors).

From the ANOVA result of Temperature vs. Temperature, the results showed that the temperature variances between sensors are significantly different throughout all periods in all pairs of compared sensors. The result is predictable since there is a difference in the temperature between the upper sensor (50 mm below surface) and lower sensor (150 mm below surface) because of the air temperature, concrete thermal conductivity, and the location of the sensors. The influence of Period 5 ANOVA, indicates there is no significant temperature difference ($F_{value} < F_{critical}$)

between most of the compared sensors, except the comparison between sensors 1 and 8 and 6 and 8. It seems there is no statistical difference between all sensors when the measured concrete temperature is greater than 20^{o}C, except for Sensor 8. Sensor 8 has a higher measured concrete temperature than the other sensors. This sensor, located on Aukland Road, has different pavement mix design than Bloor Street and this could relate to the concrete thermal conductivity.

The ANOVA result also showed that there is significant change of strain difference ($F_{value} > F_{critical}$) between sensors throughout all periods. The magnitude of change of strain depends on many factors, such as air temperature, concrete temperature, traffic load, location, moisture content, restrained and un-restrained conditions, and so on. Different sensors are located in different locations and also sustain different magnitudes of influence factors; therefore, it is reasonable that ANOVA shows the changes of concrete strain are statistically different between sensors. Continued monitoring and analysis will be carried out. It is evident from the analysis that traffic loading does impact performance.

Summary

This paper has provided a brief overview of a three way partnership which involved the installation of sensors at a City of Toronto intersection unbonded overlay. The design and construction details are presented in the paper. Traffic staging involved working over two weekends to complete the project. The pavement was instrumented in various lanes and at various depths to examine temperature and traffic impacts within the pavement. It is notable that in areas where traffic is stopped, the strain changes are less susceptible to temperature changes. In other words, the traffic appears to have the biggest impact on in-situ performance. In the case where traffic is not stopped, it is evident that temperature has a bigger impact on the change in strain. The intent in this project is to continually monitor in-situ performance. As more data is collected, mechanistic analysis will be carried out. In addition, life cycle cost analysis will be carried out to examine how feasible this rehabilitation technique is for other applications.

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