# An Integrated Field and Laboratory Research Initiative for Advancing Pavement Technology

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## ABSTRACT

New and improved pavement technologies are developed through laboratory investigations, construction and maintenance, theoretical analyses, long term performance studies such as SHRP and C-SHRP, and integrated programs of laboratory and field research. It is the latter, integrated approach, which is the subject of this paper.

In 2002, the advancement of pavement technology was given a quantum boost by the largest research funding ever provided to the field, $9 million in total. The support package has been provided from The Canada Foundation for Innovation (CFI), Ontario Innovation Trust (OIT), Ontario Research and Development Foundation (ORDCF), and several private and public sector partners. The first major initiative to occur under this support is an integrated laboratory and field research program involving new state-of-the-art testing equipment, and new expanded field and central laboratories and central and satellite field test sections.

This paper first briefly summarizes the background, partnerships, vision and focus areas for the research initiative, and then concentrates on the design, construction and performance results from the central field test sections. Particular emphasis is placed on the materials (including conventional, polymer modified, stone matrix and superpave mixes, and geogrid reinforcement), the instrumentation (including strain carriers, deflectometers, thermocouple strings and a WIM) and wireless remote monitoring, the accelerated loading (short bursts of 40 ESAL trucks, with the first 300,000 cumulative ESAL's being applied in 2 weeks), the structural evaluation (several rounds of FWD testing and layer moduli backcalculations), roughness evaluation (several rounds of IRI's) and distress evaluation. Finally, the paper offers several conclusions and recommendations as to the future opportunities for further advancements in pavement technologies.


## INTRODUCTION

### Background

The past century has seen massive gains in economic development and social advancement in Canada and many other countries. This advancement is attributable to the physical and management infrastructure for roads, airfields, buildings, water and wastewater, waste disposal, parks and recreation and various other civil facilities. Preserving the existing infrastructure asset base, and adding to it, however, poses major financial, political, environmental, resource and technological challenges.

The technological challenges in roads and pavements are particularly acute and include not only the need for asset preservation but also the provision of adequate levels of service and safety and the need for continuing innovations and advancements in all areas. It is these needs which have formed the basis for an unprecedented and new pavement research initiative. This initiative based at the University of Waterloo, Centre for Pavement and Transportation Technology (CPATT), involves a detailed research plan which involves a field testing program and construction of a test track and pavement material testing laboratory.

## Scope and Objectives

The broad scope of this paper is to describe the CPATT field test site and how it fits into the overall University of Waterloo research program. The paper focuses on the performance of the CPATT test track which examines various surface pavement types. It builds upon an earlier paper presented at the 2003 Transportation Association of Canada Conference in St. John's, Newfoundland [Tighe 2003]. The CPATT is a three way partnership which involves key players in the public, private and academic sector.

## Context

The current asset value of Canada's roads and pavements is in the order of $150 billion. Protecting this investment is of critical importance to the movement of goods and the mobility of people. Competing pressure, however, for funding from other segments of society, and having to cope with more costly and diminishing materials resources, requirements for zero-waste management and sustainability, present a real threat to our ability to protect the investment and offer the level of service expected by society. At the same time, though, there is both an opportunity and a critical need to carry out the research and technology development which will advance the planning, design, construction and operation of our roads to a new level over the coming decades [Haas 2002].

This must be accomplished through an effective and long-term partnership between researchers, public sector agencies and private industry. Accordingly our broad vision to meet the challenge involves the following key elements:

- A concentrated focus on emerging and innovative technologies.
- State-of-the-art research infrastructure comprising lab and field facilities with capability of tackling specific problems, developing new technologies and training highly qualified people (HQP).
- A substantive increase in the talent pool of HQP.
- Seeking and sustaining partnerships with individuals and organizations in technology development and applications including commercialization [Haas 2002].

The key priority areas for research at CPATT are categorized as follows:

A. Innovative structural and materials technologies for pavements.
B. Advanced computer applications related to roads.
C. Pavement construction, preservation and sustainable development.
D. Pavement and roadway safety.

Table 1 provides a summary description of these key priority areas in terms of rationale.

## THE INTEGRATED FACILITY

The overall structure of the integrated facility and research program works on a three-way partnership of public sector, private sector and universities. A Board of Directors provides general direction and priorities, with the actual execution being the responsibility of a team of researchers, technical staff and students. Since the program is centered at a university, there are certain policies, including financial and accounting procedures.

The physical home of the program is an integrated field facility and test site and a university housed laboratory. Regarding the former, it is located at the Regional Municipality of Waterloo's waste management site, which has a number of key features, not the least of which is a highly supportive municipal partner. As well, the site, which also is home to a CFI and OIT funded test track and building for fire training and research, has a land area of several hundred hectares, truck monitoring and weighing capability, access to utilities and water and close proximity (about 5 km) to the University. A building for field test equipment, repair and servicing, data acquisition units, etc. is a part of the site [Tighe 2003]

The central lab facility at the University of Waterloo incorporates state-of-the-art equipment for static and dynamic structural testing of materials, characterization of materials (e.g., SHRP) and a cold climate chamber. The latter is a particularly important aspect of the facility and is intended for testing various concrete, bituminous, geosynthetic, composite and other materials (e.g., building components) under simulated low temperature, freeze-thaw and thermal cycling conditions [Haas 2002].

Although, the initial test track is the pilot site, other satellite field test sites have already been constructed and others will continue to be constructed. For example under CPATT two projects have been placed. The first involved the construction of the instrumented concrete overlay in the city of Toronto. The second site involves the construction of three test sections which examine the noise characteristics of pavements in the Region of Waterloo. Other test sites are currently under discussion.

In addition, the CPATT is liaising with other Canadian, United States and foreign labs again where it makes mutually advantageous sense. This also involves private sector labs, and specialized facilities.

Key benefits of this initiative include the potential for full scale monitoring and testing of asphalt pavements under accelerated life cycle (torture) conditions induced by heavy truck loading. Through the evaluation of the performance and durability of an in service asphalt pavement, many new developments and potential improvements are being examined including: paving materials, mix design technology, pavement structure, construction techniques, and repair methods. Much field data collection equipment is being used to consider the effects of such factors as traffic loading and the environment. Integrated with the field site, laboratories equipped with state-of-the-art equipment and instruments allow for torture, structural, and climate testing in a controlled environment. In addition, this initiative has allowed for the assessment of geogrid reinforcement and trenchless technology. Many opportunities for new development have been created by this project which has become a training ground for many graduate and undergraduate students.

Table 1: Priority Research Areas, Rationale, Example Sub Areas and Expected Impacts [Haas 2002]

| Research Area and Rationale | Example Sub Areas | Expected Impacts |
|---|---|---|
| A. Innovative Structural and Materials Technologies for Pavements<br><br>--- changes in traffic loading --- demands for better pavement performance --- diminishing resources --- accelerated distresses --- cold climate effects --- requirements for recycling and reuse --- need for more fundamental engineering and science based technologies | a) Low temperature evaluation of materials<br>• research on engineered asphalts<br>• evaluation of new concrete mix designs and materials<br>• evaluation of new structural designs and resistance to environmental effects<br>• evaluation of recycled materials<br>b) Micro-mechanical modelling<br>• discrete element techniques to simulate particle to particle and binder interactions<br>• improved understanding of fundamental material behaviour | • Generation of substantial cost savings by minimizing premature deterioration due to cold climate effects<br>• Move toward scientific basis for materials selection and mix designs |
| B. Advanced Computer Applications Related to Roads<br><br>--- profound changes in the way of designing, building, preserving, evaluating and managing roads triggered by the computer age --- real opportunities for exploiting computer age to gain technical and economic advantages (e.g., automated surveillance technologies, diagnostic analyses, remote sensing) --- need for generating reliable, useable, data bases. | a) Instrumented test sections<br>• strain carriers, deflection gauges, moisture probes, thermistors or thermocouples, weigh-in-motion scales, etc. as required in experimental designs<br>• roadside and remote access data logging<br>b) Automated, high speed image capture<br>• use of LCD technology for image capture<br>• application of fuzzy logic and techniques such as neural networks for distress analysis and diagnostics<br>c) Intelligent Transportation Systems (ITS) applications<br>• research on roadway environmental sensing, emissions sensing and inclement weather warning systems | • Data for developing better performance models for different climatic, traffic loading and structural design conditions<br>• Data for physical distress modelling<br>• Improved consistency and reliability in data acquisition<br>• Improved marketability of Canadian developed technology and equipment<br>• Improved road safety<br>• More effective management |
| C. Pavement Construction, Preservation and Sustainable Development<br><br>--- funds required for pavement rehabilitation and maintenance are claiming major share of available budget --- great need for preservation of investment --- danger of decreasing asset value | a) Maintenance and construction methods and automation<br>• development of systematic, cost-effective procedures for pavements preservation and asset management based on reliable distress and performance data<br>• development of automated equipment and procedures for pavement maintenance and construction<br>b) New materials, recycling and waste products<br>• development of methodologies for 100% recycling and reuse of materials<br>• research on properties and performance of new and modified materials | • Moving away from traditional, reactive and worst case first maintenance<br>• Improved construction productivity and cost-effectiveness<br>• Quantum advancements toward sustainable development<br>• Becoming a leader and exporter of new technologies |
| D. Pavement and Roadway Safety<br><br>--- increased volumes and traffic density --- need to develop new and better counter measure technologies -- need to integrate technologies with non-technical factors --- | a) Research on sensing technologies<br>• pavement sensors for icing<br>• higher light reflectivity surfaces and delineations<br>b) Research on paved and partially paved shoulders<br>• safety improvements and economics | • Better warning systems<br>• New technologies with export potential |

## CPATT FIELD SITE

### Test Track Location

The CPATT commissioned the construction of a pavement test track at the Regional Municipality of Waterloo's waste management facility. Located in the southeast corner of the property, the test track runs from north to south. It is located along Erb Street West in Waterloo, the Region of Waterloo's waste management facility is within close proximity to the University of Waterloo campus making it an ideal location.

### Geotechnical Information

The Region of Waterloo is gently rolling, lying on outwash sand accompanied by sand and gravel overlaying glacial tills originating during the glacial period. From three boreholes drilled in the area of the test track, the existing material was determined to be medium to very dense and considered generally moist [Krygsman 2002]. Subgrade soils were determined to be mainly clay and sand with trace amount of gravel present. Drainage existed prior to the construction of the test track. It consisted of corrugated steel pipe culverts underneath the road bed with drainage directed easterly into a ditch which runs parallel to the test track towards a storm water management facility. Since the area is not susceptible to flooding, additional drainage was not warranted [Tighe 2003].

### Layout of the Test Track

Construction of the test track took place in June of 2002. The design and construction was expedited to take advantage of a major clay haul at the site later that summer. A total of 709 m in length and seven metres in width, this two lane test track is composed of a standard binder mix and four different surface mixes including standard Hot-Laid3 (HL3), Polymer-Modified Asphalt (PMA), Stone Mastic Asphalt (SMA) and Superpave. The binder course consisted of a standard municipal mix which was a HL4. A portion of the test track beyond 709 m was left in gravel to allow the haul vehicles a lead in to get up to speed and remove the majority of the mud from their tires before reaching the test track. The PMA section was further divided into two sections, half of which was reinforced with a BX 1200 biaxial geogrid. A diagram of the layout of these various mix designs can be seen in Figure 1. As noted, two control sections HL3-1 and HL3-2 were placed at each end of the test track [Tighe 2003].

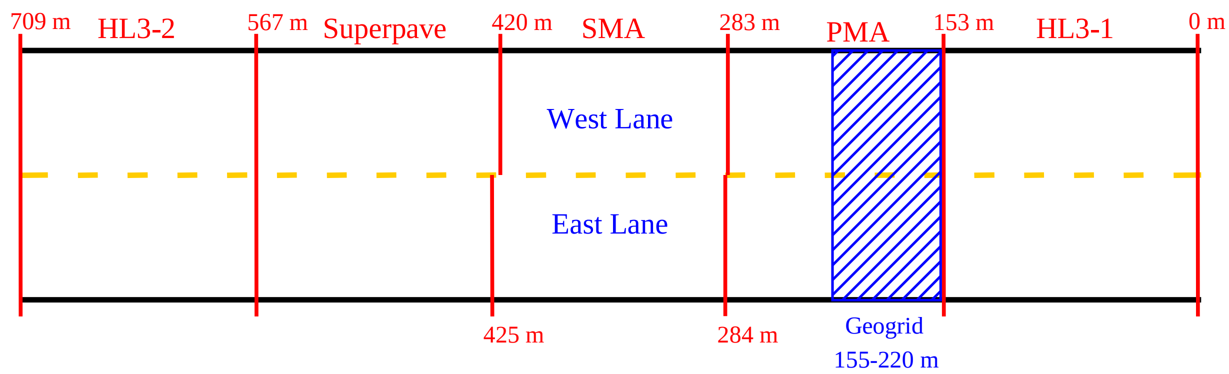


Figure 1: Layout of test track

## Test Track Construction

All materials used to construct the test track were supplied by one of two asphalt plants owned by Steed and Evans Limited. The granular 'A' as well as the HL4 binder course and HL3 surface course asphalt mixes were hauled from the S & E Heidelberg asphalt plant located at the junction of Regional Road 16 and 17 in the Region of Waterloo, northwest of the test track. All remaining asphalt mixes (Superpave, SMA, and PMA) were produced at S & E Kitchener asphalt plant located on Regional Road 6, one kilometer west of Trussler Road, also in the Region of Waterloo and south of the test track.

Taking place over the course of six working days, construction of the test track can be segregated into four stages. Stage 1 involved the placement of the granular 'A' and geogrid which was performed mainly on June 7, 2002 with a few additional loads being placed on June 10, 2002. The second stage of construction, taking place on June 11, 2002, involved the placement of a HL4 binder course over the entire length of the test track. Also occurring on June 11, 2002 as well as June 12, 2002 was stage 3 of test track construction, placement of the four different surface mixes. The final stage of test track construction involving the filling and compaction of the shoulders with granular material and took place on June 13, 2002 and June 14, 2002. June 14, 2002 also saw the extraction of core samples from all sections. A more detailed more detailed construction record can be found in [Tighe 2003].

# ANALYSIS OF DATA

## Traffic Data

In order to better understand the loading endured by the test track during the clay haul, the potential of the loading to cause damage was measured in Equivalent Single Axle Loadings (ESALs). By considering the total number of trucks of each type involved with the clay hauling, the number of days over which hauling took place, and the number of round trips on average a truck could make in a day, as well as the total ESALs from each truck, considering the contribution of all three truck axles, both loaded and unloaded it was possible to determine the

ESALs experienced by the test track. The Ontario Ministry of Transportion truck factors were used in calculating ESALs.

From data provided by the contractor performing the clay haul it was determined that the clay haul took place over eighteen days with the number of trucks being used on any one day ranging from four to nine and each truck averaging 36 runs per day. By considering manufacturer specifications for each truck type it was determined that each unloaded truck applied approximately 19 ESALs, while a loaded truck applied approximately 41 ESALs. By considering all these factors, the total ESALs endured by the test track in the summer of 2002 were calculated to be about 296,000. A second major clay haul will occur starting in May 2004. A detailed record of trucks and associated ESAL's will be monitored and the impact on the test sections will be documented.

**Material Data**

Mix designs for the SMA and Superpave asphalt mixes placed during the construction of the test track were supplied by McAsphalt Engineering Services. The HL4, HL3 and PMA mix designs were provided by Steed and Evans Limited. A break down of each mix into its components by mix percentage can be seen in Table 2. Note that the HL4 includes 19.17% Recycled Asphalt Pavement (RAP) while the SMA included 7.55% filler. SMA contains the greatest amount of virgin asphalt cement while HL4 contains the least. The HL3 and PMA have identical mix design proportions since the PMA is an HL3 with an engineered binder [McAsphalt 2002, Steed and Evans 2002].

Table 2: Mix Design Proportions

| Materials | HL4 | HL3 | PMA | SMA | Superpave |
|---|---|---|---|---|---|
| Coarse Aggregate | 36.53% | 43.70% | 43.70% | 75.47% | 48.52% |
| Fine Aggregate 1 | 40.17% | 38.00% | 38.00% | 11.32% | 46.62% |
| Fine Aggregate 2 | - | 13.30% | 13.30% | - | - |
| RAP | 19.17% | - | - | - | - |
| Filler | - | - | - | 7.55% | - |
| Virgin Asphalt Cement | 4.13% | 5.00% | 5.00% | 5.67% | 4.86% |
| PG-AC Grade | PG 58-28 | PG 58-28 | PG 70-28 | PG 70-28 | PG 70-28 |

The gradations for all mix designs used in the construction of the test track are presented in Table 3 below. With the exception of the SMA mix design, the gradations of the mixes are similar.

Table 3: Gradation Summary for Mixes

| Gradation % Passing Sieve | HL4 | HL3 | PMA | SMA | Superpave |
|---|---|---|---|---|---|
| 26.5 mm | | | | | |
| 19 mm | 100.0 | | | | |
| 16 mm | 99.3 | 100.0 | 100.0 | 100.0 | 100.0 |
| 13.2 mm | 92.7 | 98.0 | 98.0 | 98.1 | 98.8 |
| 9.5 mm | 76.9 | 80.6 | 80.6 | 69.6 | 80.6 |
| 4.75 mm | 55.1 | 54.2 | 54.2 | 24.2 | 51.0 |
| 2.36 mm | 46.9 | 44.5 | 44.5 | 21.3 | 40.6 |
| 1.18 mm | 34.6 | 31.9 | 31.9 | 18.3 | 29.7 |
| 600 µm | 21.8 | 19.8 | 19.8 | 16.2 | 22.4 |
| 300 µm | 10.5 | 10.3 | 10.3 | 13.3 | 12.5 |
| 150 µm | 4.4 | 5.2 | 5.2 | 10.6 | 5.6 |
| 75 µm | 2.0 | 3.1 | 3.1 | 8.6 | 3.0 |

Properties of each mix were also determined to insure compliance with specifications as well as to allow for comparison of the mixes. The Marshall mix design method was used for the HL4, HL3 and PMA. For the SMA and Superpave mix the Superpave mix design method was used. Properties of interest with the Superpave method include the design number of gyrations ($N_{design}$) as well as the maximum specific gravity at the initial number of gyrations (% $G_{mm}$ @ $N_{initial}$) and the maximum number of gyrations (% $G_{mm}$ @ $N_{max}$). Other properties of interest include the percentage of the total compacted volume that is air voids (Air Voids), voids in the mineral aggregate as a percentage of the total volume (VMA) and voids filled with asphalt as a percentage of the total void volume (VFA). Flow and stability of the mix are of interest with the Marshall method. In additional, the tensile strength ratio, bulk specific gravity of compacted mixture ($G_{mb}$) and maximum specific gravity of paving mixture ($G_{mm}$) were also determined. A summary of these mix properties for each mix design can be found in Table 4 [McAsphalt 2002, Steed and Evans 2002].

Table 4: Mix Design Properties

| Properties | HL4 | HL3 | PMA | SMA | Superpave |
|---|---|---|---|---|---|
| $N_{design}$ | NA | NA | NA | 100 | 125 |
| % $G_{mm}$ @ $N_{initial}$ | NA | NA | NA | 84.76 | 88.6 |
| % $G_{mm}$ @ $N_{max}$ | NA | NA | NA | 97.66 | 97.47 |
| Air Voids (%) | 4.62% | 4.62% | 4.62% | 4.00% | 4.25% |
| VMA (%) | 16.40% | 14.90% | 14.90% | 15.98% | 14.35% |
| VFA (%) | 71.83% | 71.41% | 71.41% | 74.94% | 70.39% |
| Flow (0.25 mm) | 9.6 | 9.2 | 9.2 | NA | NA |
| Stability (N) | 9500 | 8915 | 8915 | NA | NA |
| Tensile Strength Ratio (%) | NA | NA | NA | 75.20% | 73.20% |
| $G_{mb}$ - Blend | 2.359 | 2.403 | 2.403 | 2.397 | 2.416 |
| $G_{mm}$ - Blend | 2.474 | 2.510 | 2.510 | 2.454 | 2.479 |
| NA = Not Available | | | | | |

## Performance Analysis

Performance measures described herein were taken both prior to and immediately following construction. In addition measurements after the first major truck loading phase were determined and most recently measurements after the winter and prior to the second major effort in traffic loading. Key measures included Falling Weight Deflectometer (FWD) pavement load/deflection testing, International Roughness Index (IRI) surveys, distress surveys, rut depth measurements and skid numbers. The primary goal of the analysis is to critically evaluate how the various surface causes perform. In addition, the effect of traffic on the sections is evaluated in terms of key pavement performance indicators. Another aspect of this research involves the analysis of seasonal variation associated with FWD readings. This is part of the research program at the CPATT test track. However, this particular research will not be described in this paper. Instead this current paper will address current pavement performance data. Data was collected prior to paving, post construction, pre clay haul, post clay haul, pre first winter, post first winter, pre second winter and post second winter. Data from the post second winter was not available at the time this paper was submitted.

Detailed data was collected pre second winter, November 2003. This includes rut depth, IRI and friction at the test track. Information was gathered regarding the left and right wheel paths in both the west and east lanes. Four runs were made on each lane. Values were recorded every 10m in both wheel paths. The track is 709m in length; however data was only collected for 700m. HL3 1 and HL3 2 are the same type of surface but testing did not include the last 9m of HL3 2.

## FWD Analysis

FWD testing was carried out prior to construction of the test track, following construction but prior to the clay haul and following the clay haul. A Dynatest Model 8002-952 series FWD was used to measure the impact of a force, comparable to a moving tire load, exerted on the asphalt pavement surface. Measurements were taken at 12 m intervals and offsets from the pavement edge of 0.9, 2.7, 5.5, and 6.4 m. At each interval and offset, three different measured loads of magnitude 29, 40, and 53 kN were applied, while deflections were measured by seven geophones spaced at 0, 300, 600, 1200, 1500, 1800, and 2100 mm from the load centre. By adjusting values to a standard load level of 40 kN and applying the backcalculation procedure to process the FWD data, it was possible to determine the structural properties of the pavement layers and subgrade soils in terms of elastic moduli. By considering both elastic moduli and pavement thickness determined from coring results and as-built construction records it was possible to determine the resilient modulus of the subgrade ($M_R$), overall pavement modulus of elasticity ($E_P$), and effective structural number of the pavement layers ($SN_{EFF}$). A summary of these backcalculation results is presented in Table 5 [Stantec 2003].

Table 5: Summary of FWD Backcalculation Results by Section

| | | Pre Construction | | Pre Clay Haul | | Post Clay Haul | |
|---|---|---|---|---|---|---|---|
| | | Mean | St.Dev. | Mean | St.Dev. | Mean | St.Dev. |
| Resilient Modulus $M_R$ (Mpa) | HL3-1 | 41.86 | 6.62 | 51.69 | 39.37 | 43.23 | 32.63 |
| | PMA Geogrid | 45.31 | 40.92 | 42.53 | 39.84 | 39.33 | 19.89 |
| | PMA Regular | 103.31 | 36.43 | 120.95 | 26.92 | 72.02 | 17.98 |
| | SMA | 51.97 | 18.24 | 53.92 | 22.61 | 35.27 | 11.66 |
| | Superpave | 32.15 | 7.91 | 34.53 | 6.84 | 31.90 | 6.10 |
| | HL3-2 | 42.03 | 12.26 | 43.21 | 11.53 | 33.79 | 4.59 |
| Overall Pavement Modulus $E_P$ (Mpa) | HL3-1 | 155.23 | 46.73 | 247.04 | 41.03 | 414.02 | 76.57 |
| | PMA Geogrid | 84.67 | 104.04 | 150.01 | 48.26 | 322.34 | 66.34 |
| | PMA Regular | 146.07 | 34.47 | 207.74 | 22.92 | 327.72 | 46.07 |
| | SMA | 177.28 | 37.07 | 230.84 | 53.63 | 356.08 | 97.06 |
| | Superpave | 137.66 | 45.31 | 186.77 | 38.38 | 344.77 | 47.33 |
| | HL3-2 | 146.38 | 39.14 | 200.68 | 35.06 | 385.10 | 47.11 |
| Effective Structural Number $SN_{Eff}$ | HL3-1 | 2.94 | 0.40 | 3.87 | 0.23 | 4.56 | 0.30 |
| | PMA Geogrid | 1.74 | 0.34 | 3.46 | 0.33 | 4.53 | 0.32 |
| | PMA Regular | 2.29 | 0.38 | 3.82 | 0.14 | 4.43 | 0.15 |
| | SMA | 2.35 | 0.16 | 4.03 | 0.27 | 4.66 | 0.41 |
| | Superpave | 2.15 | 0.24 | 3.69 | 0.26 | 4.56 | 0.17 |
| | HL3-2 | 2.19 | 0.19 | 3.77 | 0.20 | 4.69 | 0.17 |

A graphical representation of calculated values for resilient modulus can be seen in Figure 2. Over the course of the three runs, the resilient modulus values ranged greatly, the smallest of which were measured in the PMA section reinforced with geogrid while the largest values were measured in the regular PMA section. These exceptionally high values observed in the regular PMA section may be attributed in part to the subgrade being saturated during testing, in particular, during the first two runs. With the exception of portions of the PMA and SMA sections, little variation between the three runs was noticed. In the PMA and SMA sections where a change was noticed, it was in the form of a decrease in the resilient modulus at the time of the third run performed on November 4, 2002 [Stantec 2003].

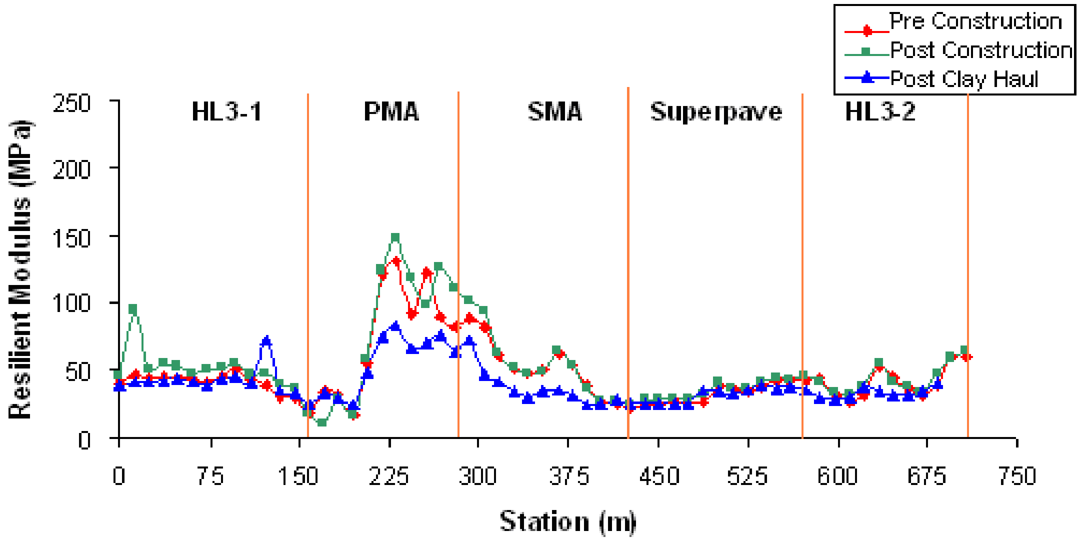


Figure 2: Subgrade resilient modulus

Figure 3 displays backcalculated results for the overall pavement modulus of elasticity for all three runs. The lowest values were measured during the first run, prior construction, in the PMA section to be reinforced with geogrid. Results from the second round of testing that took place following construction showed an increase in the pavement modulus values in all sections, indicating an increase in stiffness throughout. The PMA section reinforced with geogrid continued to have the lowest pavement modulus values. Third round testing results taken following the clay haul revealed an even greater increase in the pavement modulus values, the highest values being measured in the HL3-1 section (the exceptionally high value recorded in the SMA section was considered an outlier). In the third run, the greatest increase in pavement modulus, relative the second run, was in the PMA section reinforced with geogrid, while the least increase took place in the regular PMA and the SMA sections. Two factors that could be contributing to this increase in stiffness revealed in the third run are that the stiffness of the pavement layers increased with time or the ground was partially frozen during testing, producing misleading results [Tighe 2003].

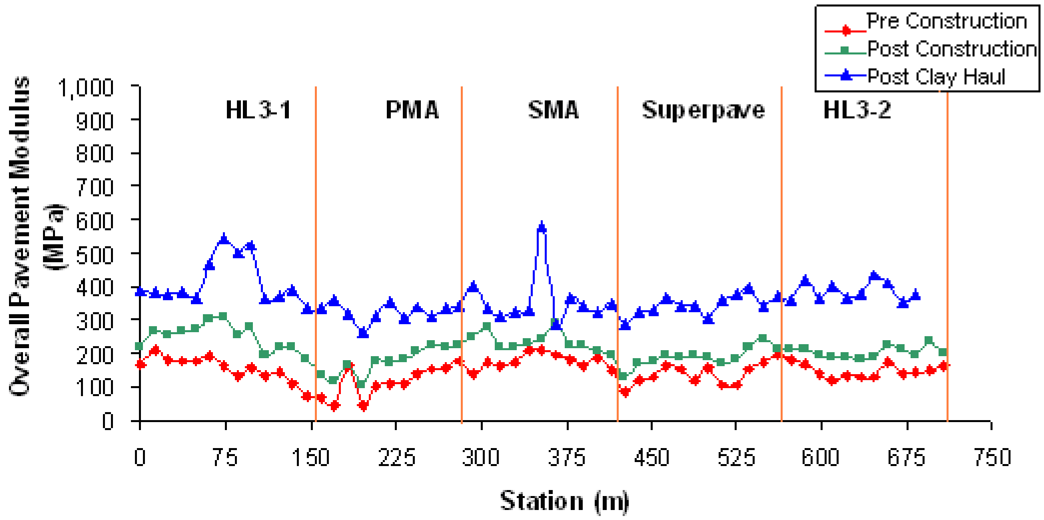


Figure 3: Overall pavement modulus of elasticity

The effective structural number of pavement layers was calculated for all section and is presented in Figure 4 for each run. A similar trend to Figure 3 is observed, with the values increasing over the course of the three runs. Once again, the lowest values for both the first and second run were recorded in the PMA section enforced with geogrid. The greatest increase in effective structural number in the third run, relative the second run, was once again observed in the reinforced PMA section while the regular PMA and SMA sections experienced the smallest increase. Little fluctuation in results was seen in the second and third runs due to little variation in material thickness, with the exception of the outlier in the SMA section where the binder material thickness was much greater [Tighe 2003].

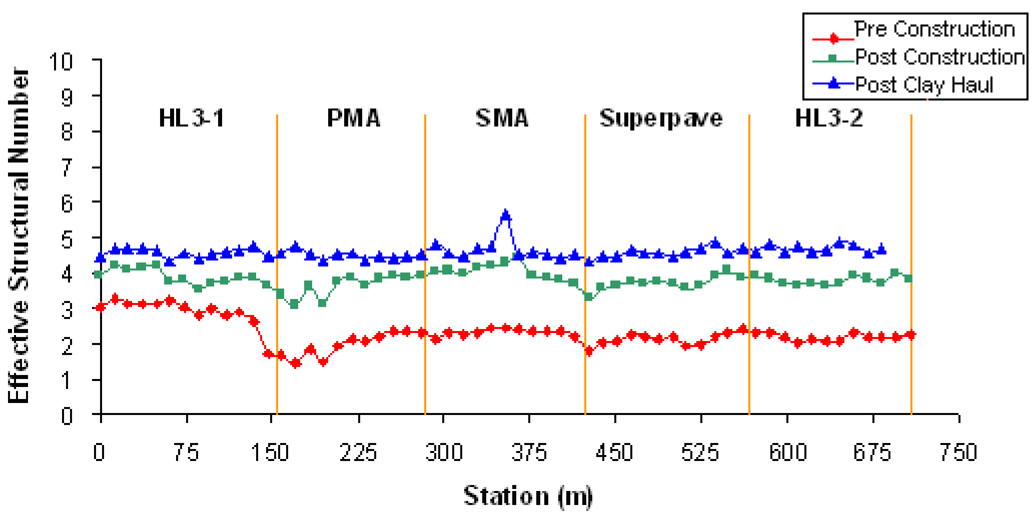


Figure 4: Effective structural number of pavement layers

Another FWD survey is planned to be performed in the spring of 2003 when it is certain the ground is thawed. This will provide results for further analysis.

**IRI Analysis**

Profiling of the roadway in order to collect roughness data was performed shortly after construction and prior to the commencement of the clay haul. Another IRI evaluation was carried out following the clay haul. A SC L009 Class I profiler, equipped with 32 kHz bumper mounted lasers was used. Three passes of each lane of the road were performed, collecting surface profiles in both the right and left wheel path at 82 mm intervals. By applying an algorithm to the surface profile data, IRI values were determined at 5 m intervals for all three passes in either wheel path of the two lanes. In order to obtain IRI values on such a small interval, it was necessary to remove the 90 m wavelength usually used in the algorithm. This portion of the algorithm is typically used to eliminate an increase in measured roughness values resulting from variation in the elevation of the roadway being profiled. It was possible to remove the 90 m wavelength from the algorithm for the analysis of this particular profiling data because of the limited variation in elevation over the length of the test track.

A field survey of the centreline of the test track revealed there to be a variation of 5.5 m in elevation over the length of the test track. These results were confirmed by applying the algorithm on a 50 m interval both with and without the 90 m wavelength. The results were compared and very little variation was noted between either technique. To acknowledge that all IRI values considered in this paper were calculated using a slightly modified algorithm (no 90 m wavelength), the notation IRI' will be used from this point forward.

To ensure the accuracy of the data collected, some additional provisions were taken. Since data collected while a profiling vehicle is getting up to speed are inaccurate, the first 50 m of each run was dismissed as unacceptable data. Note these sections are both control sections consisting of HL3 surface course over HL4 binder course. As well, to insure accurate division of the data into respective sections, the data contained in a small transition zone between sections was eliminated. This measure also insured that roughness associated with the joint between sections did not impact results

Figure 5 compares the average IRI' values calculated using the profile data collected during the three runs. A sharp increase in IRI' values following the clay haul is visible from approximately 150 to 225 m in the PMA section reinforced with geogrid. As well, smaller sharp increases are also apparent from about 430 to 450 m and 510 to 530 m in the Superpave section. Although it is difficult to be certain from Figure 5, it appears as though there is little change in the IRI' values in the remaining sections over the course of the three runs.

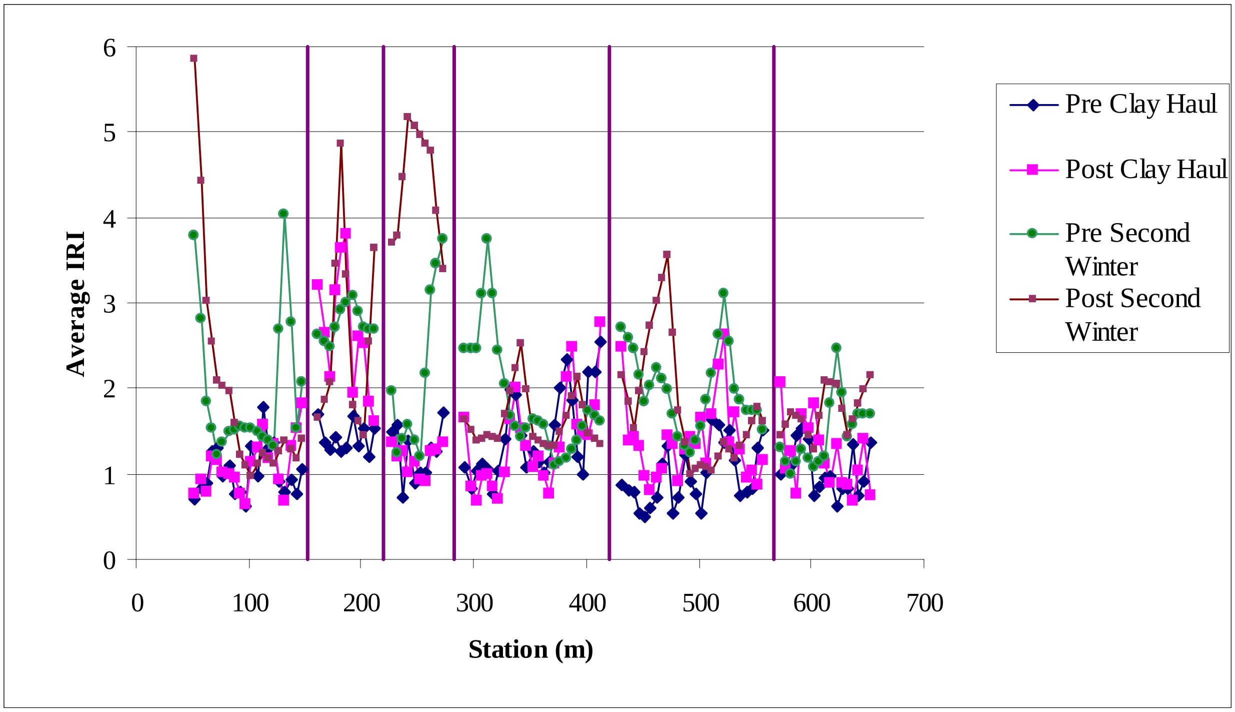


Figure 5: IRI' values for the four test dates

To obtain a better understanding of the IRI' data obtained from the test track, Analysis of Variance (ANOVA) was performed on the IRI' results. First, to insure consistency between lanes, ANOVA was carried out comparing the IRI' values determined for the eastbound lane to the westbound lane for each of the sections. If the $F_{Calculated}$ value was determined to be smaller than the $F_{Critical,}$ then the lanes are statistically the same and vice versa. A summary of the results can be seen in Table 6. IRI' values calculated for the east and west lanes of each section were statistically the same on the initial IRI evaluation. With the exception of the Superpave section, the IRI' values calculated after the clay haul were shown to be statistically the same. The reason for the IRI' values for the two Superpave lanes not being the same may relate to one lane experiencing more deterioration than the other lane due to a weaker subgrade as well as a variety of other factors. Since for the most part it appears as though the IRI' values calculated for the eastbound and westbound lanes are statistically the same, it is reasonable to combine them and treat them as a whole.

Table 6: IRI' ANOVA Results for East Lane vs. West Lane

| | Pre Clay Haul | | Post Clay Haul | | Pre Second Winter | | Jul-04 | |
|---|---|---|---|---|---|---|---|---|
| | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ |
| **HL3-1** | 0.5 | 4.1 | 2.2 | 4.1 | 0.8 | 4.6 | 1.1 | 4.6 |
| **PMA Geogrid** | 2.5 | 4.4 | 0.0 | 4.4 | 3.5 | 6.0 | 0.4 | 6.3 |
| **PMA Regular** | 1.5 | 4.4 | 2.9 | 4.4 | 3.0 | 6.6 | 2.9 | 6.3 |
| **SMA** | 1.1 | 4.1 | 0.7 | 4.1 | 2.0 | 4.6 | 5.0 | 4.7 |
| **Superpave** | 1.3 | 4.0 | 7.9 | 4.0 | 2.4 | 4.7 | 4.5 | 4.6 |
| **HL3-2** | 2.9 | 4.1 | 0.2 | 4.1 | 1.2 | 4.7 | 3.7 | 5.2 |

To ensure the accuracy of the observations made from Figure 5, ANOVA was used to compare the IRI' values collected prior the clay haul to those collected after. A summary of these results is shown in Table 7. As previously observed from Figure 5, both the PMA section reinforced with geogrid and the Superpave section are not statistically the same. Thus, these two sections experienced the greatest change in roughness over the course of the clay haul. The IRI' values for the remaining section were determined to be statistically the same before and after the clay haul indicating limited impact on the roughness of these sections from the loading.

Table 7: IRI' ANOVA Results for Pre Clay Haul vs. Post Clay Haul and Post Clay Haul vs. Pre Second Winter

| | Post Clay Haul | | Pre Second Winter | |
|---|---|---|---|---|
| | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ |
| **HL3-1** | 0.7 | 4.0 | 1.1 | 4.6 |
| **PMA Geogrid** | 18.8 | 4.1 | 12.7 | 6.0 |
| **PMA Regular** | 0.3 | 4.1 | 3.7 | 6.6 |
| **SMA** | 0.4 | 3.9 | 0.8 | 4.7 |
| **Superpave** | 11.5 | 3.9 | 2.1 | 4.6 |
| **HL3-2** | 3.0 | 4.0 | 10.4 | 4.8 |

Although it has been shown that some surface materials reacted differently than others to loading, comparison of the various surface materials were carried out. This ANOVA involved comparing the IRI' results for the two HL3 section to ensure the two control sections were statistically the same. Once this was established as shown in Table 8 for both before and after the clay haul, the combined IRI' results for the HL3 sections were compared to the various other surface materials. A summary of these results can also be found in Table 8. As would be expected, the IRI' values collected from the two HL3 sections were statistically the same on both profiling dates. However, pre clay haul the IRI' values collected from both the reinforced PMA and the SMA were not statistically the same as those collected from the HL3 sections. Post clay haul, in addition to the sections not statistically the same on the previous profiling data, the IRI' values for the Superpave section were also not statistically the same as those from the HL3 sections. These results tell us that the roughness of these surface materials were differences, especially over time.

Table 8: IRI' ANOVA Results for HL3 (Control Sections) vs. Surface Test Sections

| | Pre Clay Haul | | Post Clay Haul | | Pre Second Winter | |
|---|---|---|---|---|---|---|
| | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ | $F_{Calculated}$ | $F_{Critical}$ |
| **HL3-1 vs. HL3-2** | 0.1 | 4.0 | 1.0 | 4.0 | 4.6 | 4.7 |
| **HL3-1 vs. PMA Geogrid** | 11.3 | 3.9 | 77.5 | 3.9 | 0.0 | 6.0 |
| **HL3-1 vs. PMA Regular** | 2.5 | 3.9 | 0.4 | 4.0 | 0.3 | 6.6 |
| **HL3-1 vs. SMA** | 18.6 | 3.9 | 7.6 | 3.9 | 6.1 | 4.7 |
| **HL3-1 vs. Superpave** | 1.5 | 3.9 | 6.2 | 3.9 | 1.5 | 4.6 |

Table 9 outlines the average IRI for each lane and wheel path. The average for each surface type was then calculated. The Superpave section has the smallest average IRI but is close to HL3 2 and HL3 1. The PMA section has the largest average IRI being almost 0.7 larger than the next. The PMA section IRI is the largest in every wheel path.

Table 9: Average IRI for Each Wheel Path and Lane

| | Left | | Right | | Average |
|---|---|---|---|---|---|
| **Surface Type** | *West* | *East* | *West* | *East* | |
| ***HL3-1*** | 2.5 | 1.9 | 2.1 | 1.7 | 2.0 |
| ***PMA regular*** | 1.8 | 1.6 | 2.1 | 1.7 | 1.8 |
| ***PMA Geogrid*** | 3.1 | 3.3 | 3.4 | 3.5 | 3.3 |
| ***SMA*** | 1.5 | 1.8 | 1.6 | 2.5 | 1.9 |
| ***Superpave*** | 1.3 | 1.6 | 1.5 | 2.2 | 1.6 |
| ***HL3-2*** | 1.3 | 2.3 | 1.4 | 2.0 | 1.8 |

Table 10 summarizes the ANOVA for the IRI values of the left verses right wheel paths in both lanes. The values of the west and east lanes are also compared. The calculated and critical numbers where found for every run and then averaged.

The left and right wheels of SMA are statistically different based on the F test. Similar to SMA, the right and left wheels of Superpave are statistically different. This indicated that the IRI in the left wheel path is different from the right wheel path.

In the west lane of the PMA section and the Superpave section, the ANOVA shows a much higher calculated F value than the other surfaces with PMA's being slightly higher. In the east lane HL3 1 has the largest calculated number of the statistically comparable surfaces.

Table 10: Comparison of IRI of Left vs. Right Wheel Paths in West and East Lanes

| **Lane** | **HL3-1** | | **PMA** | | **SMA** | | **Superpave** | | **HL3-2** | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit |
| **West** | 0.45 | 4.60 | 2.83 | 4.84 | 0.26 | 4.60 | 2.41 | 4.60 | 1.00 | 4.75 |
| **East** | 2.18 | 4.60 | 0.27 | 4.75 | 14.06 | 4.60 | 10.53 | 4.67 | 0.63 | 4.75 |

The SMA and Superpave sections of the east lane are statistically different for the left and right wheel path.

**Distress Surveys**

Various visual distress surveys are performed on a regular basis. These include post construction, pre clay haul, post clay haul, post first winter, pre second winter, post second winter, pre second clay haul. Following construction of the test track fat spots were observed in random locations throughout the SMA, however, no other distresses were visible at this time. Once the clay haul was complete more distresses had begun to appear. These distresses included segregation in portions of the Superpave section and raveling in some parts of the SMA. As well, faint cracking was noticed in the PMA section reinforced with geogrid, while a very noticeable road deformation had begun to form in the Reinforced PMA section and the HL3-1 section. Some correspondence is noticeable between IRI and distress survey results.

Post first winter, there was a limited progression of distresses. However, there was a notable subgrade failure in the geogrid section. This was repaired prior to the second winter. During an evaluation post second winter and pre second clay haul. Overall, the SMA is performing the best in terms of exhibiting the least amount of distress at this time. There is some cracking shown in the other sections namely the HL3 1.

**Rut Analysis**

Table 11 shows data from the November 2003 evaluation. It compares the average rut depths for each wheel path in the different sections as well as the standard deviation.

Table 11: Average Rut Depth (mm) and Standard Deviation for Pre Second Winter Survey

| | **Left** | | **Right** | | **Average** | **Standard Deviation** |
|---|---|---|---|---|---|---|
| **Surface Type** | *West* | *East* | *West* | *East* | | |
| ***HL3 1*** | 2.8 | 2.4 | 2.7 | 2.7 | 2.7 | 0.1 |
| ***PMA Geogrid*** | 2.6 | 3.3 | 2.4 | 2.6 | 2.7 | 0.4 |
| ***PMA Regular*** | 2.9 | 3.0 | 2.4 | 2.7 | 2.7 | 0.2 |
| ***SMA*** | 3.2 | 3.3 | 3.3 | 2.9 | 3.1 | 0.2 |
| ***Superpave*** | 2.6 | 2.3 | 2.8 | 2.6 | 2.6 | 0.2 |
| ***HL3 2*** | 2.3 | 2.5 | 2.5 | 2.8 | 2.5 | 0.2 |

Table 11 shows the average rut depth for each type of surface. SMA has the largest average depth while HL3-2 has the smallest average depth.

Table 12 compares the rut depths of the left verses the right wheel paths in each of the five sections for both the east and west lanes. To ensure the greatest level of accuracy, an ANOVA was carried out. Each run was first computed to ensure that the differences in the four separate runs were not statistically significant. Once this was confirmed then average values were used for analysis.

HL3-1 and HL3-2 in the east and west lanes were calculated and it was determined that there are no statistical differences between the two lanes. In other words the east and west lanes are the same.

The Superpave test section resulted with the largest Fcalculated number in the west lane while SMA is the largest in the east lane. HL3-1 has the smallest calculated value in both the west and east lanes, however none of the Fcalculated values exceed Fcritical, therefore the left and right wheel paths are statistically the same.

Table 12: Comparison of Rut Depths of the Left vs. Right Wheel Paths in West and East Lanes

| **Lane** | **HL3-1** | | **PMA** | | **SMA** | | **Superpave** | | **HL3-2** | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit | Fcalc | Fcrit |
| **West** | 0.97 | 4.71 | 2.55 | 4.75 | 1.69 | 4.60 | 2.76 | 4.67 | 1.26 | 4.67 |
| **East** | 0.67 | 4.54 | 1.48 | 4.67 | 2.40 | 4.60 | 2.03 | 4.67 | 1.20 | 4.75 |

**Friction Analysis**

Table 13 provides the friction numbers over each type of surface in the two lanes. When trucks are traveling north they are in the east lane and when they are going south they are in the west lane. When vehicles move north they are moving a full load of clay out of the site. However, when they travel south the vehicle is empty. The friction numbers show that there is a correlation between the mass of the load and the friction number. In all cases, with the exception of the PMA, the friction number is greater when a smaller load is present. PMA does not show any change between the load sizes. SMA shows only a minimal change while HL3-2 has the greatest differences. HL3-1 and HL3-2 show the same friction number for the southbound lane which is ideal as they are the same surface type.

Table 13: Friction Numbers of the Surface Types in Each Lane

| **Mix Type** | **Direction** | **Friction Number 50** | **Field** | **Range** |
|---|---|---|---|---|
| ***HL3-2*** | N | 61 | 3 | 60-63 |
| | S | 70 | 2 | 69-71 |
| ***Superpave*** | N | 74 | 4 | 71-77 |
| | S | 77 | 4 | 72-79 |
| ***SMA*** | N | 70 | 4 | 68-72 |
| | S | 71 | 3 | 68-72 |
| ***PMA*** | N | 72 | 3 | 67-75 |
| | S | 72 | 4 | 70-74 |
| ***HL3-1*** | N | 67 | 3 | 64-68 |
| | S | 70 | 4 | 64-75 |

## Comments on Recent Evaluations

HL3-1 and HL3-2 are the same surface type and were shown to perform similarly throughout the testing. HL3-2 had a small average rut depth and the depth of HL3-1 was only slightly larger. HL3-1 however had the smallest standard of deviation for rut depth. The average IRI values of HL3-1 and HL3-2 were very similar and average amongst all the surface types. Both HL3-1 and HL3-2 had a friction number of 70 in the southbound lane although HL3-2 had a friction number much smaller than that of HL3-1 in the northbound lane.

To date the trial mixes are all performing well, the rut depths are negligible at this point at 4mm. In addition all of the skid values are very good, there are slight differences in the IRI. However, these will continue to be monitored.

## CONCLUSIONS

This paper has described a new initiative in roads and pavement research involving an integrated laboratory and field facility and an unprecedented level of federal, provincial, municipal, private sector and university support to create the necessary infrastructure.

A context for the research initiative has been established and it focuses on a number of key issues, the driving forces which impact on these issues, and the resulting opportunities and future prospects. Within this context, a broad vision has been formulated with the following major elements: emerging and innovative technologies, state-of-the-art research infrastructure, increasing the talent pool of highly qualified people and establishing sustained partnerships with the public and private sectors and the universities.

An overall structure for the integrated facility and program are described, as well as examples of the key, priority areas within the program. These areas have been grouped as follows: (A) Innovative structural and materials technologies, (B) Advanced computer applications related to roads, (C) Pavement construction, preservation and sustainable development and (D) Pavement and roadway safety. As well, the philosophy and required skill sets underlying the formation of the research team have been described.

The design parameters and construction data from the UW CPATT test track have been presented in this paper. Initial results from IRI, deflection and distress surveys have also been presented and analyzed. Monitoring of the test track will continue over time and this data will be used to develop performance models for the various asphalt mixes.

The SMA, Superpave and PMA mixes are showing superior performance as compared to the traditional HL3 mix. CPATT will continue to monitor performance over time. Data will continue to be analyzed and used in the training and education of both undergraduate and graduate students.

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