Mechanistic-Empirical Design and Design Validation

Toronto Highway 407
East Partial and Western Extension Freeway Design

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ABSTRACT

In 1999, the Ontario Ministry of Transportation awarded the Highway 407 ETR Concession a 99-year lease to operate, maintain, and expand Ontario Highway 407. Highway 407 consists of 69 km of 6-lane divided highway which extends in an east/west direction north of Toronto, Ontario. The current expansion of Highway 407 includes extensions to the east (14.7 km – to Markham) and west (24.7 km to Hamilton).

The initial designs for the Highway 407 pavement structures were completed using the 1993 AASHTO *Guide for Design of Pavement Structures* and the associated DARWin pavement design software. After completing the AASHTO-based designs, a mechanistic analysis of each design section was completed using the Shell BISAR computerized elastic-layer pavement analysis program and an established asphalt concrete fatigue damage model. BISAR facilitates the mechanistic modelling of the pavement structure and calculates the stresses and stains caused by vehicle loading. The purpose of the mechanistic analysis was to predict when pavement structural deterioration (fatigue cracking) would reach a level that the pavement required structural enhancement (e.g., an HMA overlay). The critical pavement response is the horizontal (tensile) strain at the bottom of the asphalt layer. The tensile strain occurs when the pavement is loaded and is a function of the load magnitude, tire pressure, and pavement thickness and stiffness. Over time, repeated tensile strains at the bottom of the AC layer result in fatigue cracks that begin at the bottom of the HMA layer and propagate to the pavement surface.

Each pavement section was modeled in BISAR using the pavement layer thickness and the material properties developed for the AASHTO designs. The structural characteristics of the subgrade and pavement layers vary from month to month, depending on moisture conditions and temperature. After completing the BISAR analysis, the monthly AC stiffness (modulus) values and associated tensile strains were used as inputs into a fatigue equation from the program PDMAP. The PDMAP equation is based on laboratory test data that has been calibrated to the levels of cracking observed at the AASHO Road Test.

This paper presents an outline of the analysis, calibration, and validation methodology for the mechanistic-empirical pavement design of the high-volume Highway 407.

INTRODUCTION

Highway pavement design in the Province of Ontario is generally based on the *Ontario Pavement Rehabilitation and Design Guide* [*1*]. Pavement structural design is generally carried out using the Ontario Ministry of Transportation (MTO) “empirical” design procedure. In this procedure, the structural contribution of each of the individual pavement layers are transformed into an equivalent thickness of granular base material; for example, 25 mm of hot mix asphalt (HMA) is equivalent to 50 mm of granular base is equivalent to 75 mm of granular subbase. The required total granular base equivalency is then compared to the required granular base equivalency for a given subgrade soil type and traffic level. The design procedure is based on past pavement design experience. Although the procedure is relatively simple, it does not allow the designer to assess critical pavement performance parameters such as asphalt concrete fatigue and pavement structure rutting, nor does it allow specific evaluation of traffic/pavement interaction.

The *Pavement Rehabilitation and Design Guide* also permits the use of the 1993 AASHTO Pavement Design Guide [*2*] procedures. The AASHTO Guide is one of the most widely used pavement design procedures in North America. The selection of specific design parameters is based on the knowledge of local materials and construction practices, traffic loads, and environmental conditions.

The initial designs for the Highway 407 pavement structures were completed using the 1993 AASHTO *Guide for Design of Pavement Structures* and the associated DARWin [*3*] pavement design software. After completing the AASHTO-based designs, a mechanistic analysis of each design section was performed using the BISAR computerized elastic-layer pavement analysis program and an established asphalt concrete fatigue damage model. The purpose of the mechanistic analysis was to predict when pavement structural deterioration (fatigue cracking) would reach a level that the pavement required structural enhancement (e.g., an HMA overlay).

Each pavement section was modeled in BISAR [*4*] using the pavement layer thickness and the material properties developed for the AASHTO designs. The structural characteristics of the subgrade and pavement layers vary from month to month, depending on moisture conditions and temperature. After completing the BISAR analysis, the monthly HMA stiffness (modulus) values and associated tensile strains were used as inputs into a fatigue equation from the program PDMAP [*5*]. The PDMAP equation is based on laboratory test data that has been calibrated to the levels of cracking observed at the AASHO Road Test.

The ultimate performance of a highway pavement depends on many factors, including environment, traffic and the quality of construction. While it is possible to develop pavement designs to account for these factors, the accuracy of the design will depend strongly on the validity of the subgrade, pavement material, and traffic assumptions. In order to validate the pavement design assumptions for the Highway 407 pavement, Falling Weight Deflectometer (FWD) pavement load/deflection testing was completed.

PAVEMENT DESIGN METHODOLOGY

Initially, the pavement structures were designed using the AASHTO 1993 procedures. These pavement designs were then checked using a mechanistic analysis. The traffic data, subgrade modulus, structural and drainage layer coefficients, and material moduli were determined based on field and laboratory test data as well as local knowledge. A summary of the pavement layer parameters used for the AASHTO design is given in Table 1.

The effect of traffic loads on a pavement is expressed using the concept of axle Load Equivalency Factors (LEF) for each vehicle axle that traverses the pavement. The LEF is related to the damage expected to be caused by a standard load of 80 kN carried by a single axle with dual tires. Each LEF for each vehicle is cumulated and summarized to develop the number of Equivalent Single Axle Loads (ESALs) the pavement is expected to carry over its design life.

Table 1. Highway 407 AASHTO Pavement Layer Design Parameters.

Pavement Element	**AASHTO Pavement Design Parameters**	
	Structural (a_i)	**Drainage (C_d)**
Hot Mix Asphalt	0.44	1
Open Graded Drainage Layer	0.14	1.15
Granular A Modified (base)	0.14	0.95
Granular A Sand and Gravel (base)	0.11	0.95
Granular B Type I Subbase	0.09	0.95

In the development of design ESAL data for pavement design, it is necessary to consider any site-specific traffic survey data. If detailed traffic and axle load data is not available, it is necessary to determine ESALs based on the percentage of commercial vehicles, distribution of commercial vehicles into truck classes, truck factors for different truck classes, and the lane distribution factors. The truck factors (number of ESALs per truck) used for the Highway 407 traffic analysis are given in Table 2.

Table 2. Truck Factors used for Highway 407 ESAL Calculations.

Major Truck Class	**Truck Factor**	**Range of Typical Truck Factors**
2- and 3-axle trucks	0.5	0.05 to 1.0
4-axle trucks	2.3	0.2 to 4.0
5-axle trucks	1.6	0.3 to 3.5
6 and more axle trucks	5.5	2.0 to 7.0

The ability of subgrade soil to support a pavement structure is characterized by its laboratory-determined resilient modulus, M_R. The AASHTO design equation for flexible pavements is very sensitive to this input.

The Ontario Ministry of Transportation recently published guidelines for the categorization of Ontario subgrade soils [*6*]. Subgrade types are grouped into eight categories, and each category is further subdivided into soils in good, fair, and poor condition, with M_R values assigned to each grouping and soil condition as shown in Table 3.

The resilient modulus to be used for the AASHTO-based designs is a laboratory-determined M_R for springtime conditions. The AASHTO design equation for flexible pavements has been calibrated to reflect the M_R that existed at the AASHO Road Test. Consequently, the resilient modulus used for a given roadbed soil must be referenced to the resilient modulus of the A-6 (yellow-brown clay) roadbed soil at the AASHO Road Test, which had M_R of 21 MPa.

Based on geotechnical field and laboratory investigations, design spring modulus values of 25 MPa and 50 MPa were selected for the East Partial and Western Extensions, respectively. Other relevant AASHTO pavement design procedure details are given in Table 4.

Table 3. Recommended M_R Values for the AASHTO–Ontario Pavement Design Model.

Brief Description	**MTO Classification (MTO, 1980)**	**Drainage Characteristics**	**Susceptibility to Frost Action**	**Resilient Modulus (M_R) for Typical Subgrade Conditions, MPa**		
				Good	**Fair**	**Poor**
Rock, rock fill, shattered rock, boulders/cobbles	Boulders/ cobbles	Excellent	None	90	80	70
Well graded gravels and sands suitable as granular borrow	GW, SW	Excellent	Negligible	80	70	50
Poorly graded gravels and sands	GP, SP	Excellent to fair	Negligible to slight	70	50	35
Silty gravels and sands	GM, SM	Fair to semi-impervious	Slight to moderate	50	35	30
Clayey gravels and sands	GC, SC	Practically impervious	Negligible to slight	40	30	25
Silts and sandy silts	ML, MI	Typically poor	Severe	30	25	18
Low plasticity clays and compressible silts	CL, MH	Practically impervious	Slight to severe	35	20	15
Medium to high plasticity clays	CI; CH	Semi-impervious to impervious	Negligible to severe	30	20	15

Table 4. AASHTO Pavement Design Parameters.

Design Parameter	**407 Western Extension**	**407 East Partial Extension**
Design ESALs (15 years)	20,000,000	13,700,000
Serviceability Loss	2	2
Reliability (%)	90	90
Standard Deviation	0.45	0.45
Spring Subgrade Modulus (MPa)	25	50

The serviceability loss is the difference in the condition of the pavement from when it is constructed until it is rehabilitated. The reliability of a pavement design-performance is the probability that a pavement section design using the process will perform satisfactorily over the traffic and environmental conditions for the design period. The standard deviation takes into account the variability associated with design and construction inputs, including variability and material properties, subgrade, traffic and environmental exposure. The selected values, as outlined in Table 4 are those typically used for the design of freeway facilities of this type.

The AASHTO structural number (SN) and allowable ESALs were calculated for each pavement. The allowable ESALs were then compared to the estimated ESALs based on the traffic analysis to determine the expected design life of the pavement in years.

After completing the AASHTO-based designs, a mechanistic analysis was performed for each design section using the BISAR computerized elastic-layer pavement analysis program and an established asphalt concrete fatigue damage model. The purpose of the mechanistic analysis was to predict when pavement structural deterioration (fatigue cracking) would reach a level that the pavement required structural enhancement (e.g., an HMA overlay). Ideally, an HMA pavement designed for a 15-year initial service period will provide at least 10 years of service before an overlay is needed. It is estimated that the HMA will reach 10 percent fatigue cracking in the wheel paths at about year 10. Selective machine patching or an overlay would then be completed to remove the negative effects of aging, weathering and rutting.

The critical pavement response is the horizontal (tensile) strain at the bottom of the asphalt layer. The tensile strain occurs when the pavement is loaded and is a function of the load magnitude, tire pressure, and pavement thickness and stiffness. Over time, repeated tensile strains at the bottom of the HMA layer result in fatigue cracks that begin at the bottom of the HMA layer and propagate to the pavement surface.

Each pavement section was modeled using the Shell BISAR programming using the proposed pavement layer thickness and the material properties. BISAR facilitates the mechanistic modelling of the pavement structure and calculates the stresses and stains caused by vehicle loading.

An example of the structural analysis for the Highway 407 East Partial Extension is shown in Table 5. As noted in the table, the structural characteristics of the subgrade and pavement layers vary from month to month, depending on moisture conditions and temperature. After modeling the pavement section, two different single axle wheel loads were simulated (80 kN and 106 kN) using BISAR, and the corresponding critical HMA tensile strains were calculated for each month of the year. The 80-kN load represents the standard ESAL, and the 106-kN load represents an overloaded truck (it was assumed that 90 percent of the truck loads were legal and that 10 percent were overloaded). After completing the BISAR analysis, the monthly HMA stiffness (modulus) values and associated tensile strains were used as inputs into a fatigue equation from the program PDMAP. The PDMAP equation is based on laboratory test data that has been calibrated to the levels of cracking observed at the AASHO Road Test. A summary of the equation is as follows:

$$\text{Log}\,(N_{all}) = 15.947 - 3.291 \times \text{Log}\,(\text{AC Strain}) - 0.854 \times \text{Log}\,(E_{ac})$$

where,

N_{all} = Number of allowable applications until 10 percent HMA fatigue cracking in the wheel path.
AC Strain = Horizontal microstrain at the bottom of the asphalt layer, in/in x 10^6.
E_{ac} = Asphalt modulus, ksi.

The PDMAP equation estimates the number of allowable axle loads that the pavement can withstand before 10 percent of the wheel path develops asphalt fatigue cracking. The fatigue damage for each month is then calculated as the ratio of the expected loadings over allowable loadings, and the total fatigue damage is the sum of the monthly fatigue damage values. To complete the analysis, the number of expected loadings was varied until a total fatigue damage of 1.0 was achieved. The resulting traffic level was then compared to the traffic projections to estimate when the pavement would exceed the threshold of 10 percent fatigue cracking in the wheel paths.

Note that the PDMAP model only provides an approximate estimate of the time it will take for fatigue cracking to develop along 10 percent of the wheel path. Actual material properties, climatic conditions, and traffic loads will greatly affect the rate of crack development, and some sections may fail more rapidly than predicted while other sections may perform much better than predicted. An example mechanistic analysis for Highway 407 East Partial Extension is given in Table 5.

A summary of the designs for the Highway 407 East Partial and Western Extensions is given in Table 6.

Table 6. Summary of Highway 407 Main Lane Pavement Designs.

Item	**Western Extension**	**East Partial Extension**
Design Traffic ESALs (15 years)	20,000,000	13,700,000
Dense Friction Course (mm)[1]	40	40
Heavy Duty Binder Course (mm)	80	80
HL 8 Binder Course (mm)	100	120
Asphalt Treated Open Graded Drainage Layer (mm)	100	0
Granular A Modified Baser (mm)	360	400
Total Thickness (mm)	680	640
AASHTO Structural Number (mm)	161	160
AASHTO Allowable ESALs	22,000,000	14,000,000
AASHTO Design Life (years)	16.5	15.5
ESALs Until 10 Percent Fatigue Cracking	13,500,000	9,340,000
Estimated First Year of Overlay	10	10.5

1. Dense Friction Course (DFC) is a high quality durable surface course mix used for high volume highway pavements in Ontario.
2. Heavy Duty Binder Course (HDBC) is a high quality rut resistant binder course.
3. HL 8 is a standard binder course asphalt generally used for low layers in relatively thick asphalt pavement structures.
4. Asphalt treated open graded drainage layer (AC OGDL) is a permeable base layer.
5. Granular A Modified is a 100 percent crushed particle, high quality dense granular base.

The western extension has a slightly thicker pavement structure than the east partial extension due to the lower bearing capacity soils in the western extension area and somewhat higher traffic levels in the west. The western extension connects major urban centres (Toronto and Hamilton) and also serves as a part of a major U.S./Canadian transportation corridor.

Table 5. Example Mechanistic Analysis for Highway 407 East Partial Extension.

Standard Load (80 kN)

Month	n	AC thickness,	Subbase thickness,	AC stiffness (Eac),	Granular subbase stiffness (Esbase)	Subgrade stiffness (Esubgrade),	Frozen Subgrade		Strain	Traffic	N (PDMAP-10)	Damage
		Mm	mm	GPa	MPa	MPa	Thickness	E, MPa				
Jan	1	240	400	13.8	265	103	600 mm	345	3.54E-05	0.70	1.072E+08	0.005
Feb	1	240	400	13.8	265	103	600 mm	345	3.54E-05	0.70	1.072E+08	0.005
Mar	1	240	400	10.3	199	52			5.11E-05	0.70	4.095E+07	0.012
Apr	1	240	400	5.2	132	52			9.24E-05	0.70	1.054E+07	0.047
May	1	240	400	4.0	132	69			1.10E-04	0.70	7.393E+06	0.068
Jun	1	240	400	2.4	165	83			1.55E-04	0.70	3.681E+06	0.136
Jul	1	240	400	1.7	165	103			1.95E-04	0.70	2.305E+06	0.217
Aug	1	240	400	2.1	165	103			1.70E-04	0.70	3.098E+06	0.161
Sep	1	240	400	3.1	199	103			1.22E-04	0.70	6.530E+06	0.077
Oct	1	240	400	5.2	199	103			8.22E-05	0.70	1.548E+07	0.032
Nov	1	240	400	5.2	199	103			8.22E-05	0.70	1.548E+07	0.032
Dec	1	240	400	12.4	265	103	600 mm	345	3.85E-05	0.70	8.897E+07	0.006
Total										**8.41**		**0.798**

Overload (106 kN)

Month	n	AC thickness,	Subbase thickness,	AC stiffness (Eac),	Granular subbase stiffness (Esbase)	Subgrade stiffness (Esubgrade),	Frozen Subgrade		Strain	Traffic	N (PDMAP-10)	Damage
		Mm	Mm	GPa	MPa	MPa	Thickness	E, MPa				
Jan	1	240	400	13.8	265	103	600 mm	345	4.60E-05	0.08	4.527E+07	0.001
Feb	1	240	400	13.8	265	103	600 mm	345	4.60E-05	0.08	4.527E+07	0.001
Mar	1	240	400	10.3	199	52			6.65E-05	0.08	1.721E+07	0.003
Apr	1	240	400	5.2	132	52			1.20E-04	0.08	4.458E+06	0.012
May	1	240	400	4.0	132	69			1.42E-04	0.08	3.190E+06	0.017
Jun	1	240	400	2.4	165	83			2.00E-04	0.08	1.591E+06	0.035
Jul	1	240	400	1.7	165	103			2.51E-04	0.08	1.004E+06	0.055
Aug	1	240	400	2.1	165	103			2.19E-04	0.08	1.346E+06	0.041
Sep	1	240	400	3.1	199	103			1.57E-04	0.08	2.847E+06	0.020
Oct	1	240	400	5.2	199	103			1.06E-04	0.08	6.705E+06	0.008
Nov	1	240	400	5.2	199	103			1.06E-04	0.08	6.705E+06	0.008
Dec	1	240	400	12.4	265	103	600 mm	345	4.99E-05	0.08	3.789E+07	0.001
Total										**0.93**		**0.205**

Total ESAL= **9.34** Total Fatigue Damage = **1.0029**

DESIGN VALIDATION

To validate the pavement designs, the actual pavement response to loading was determine using a Dynatest 8081 High Capacity Falling Weight Deflectometer. In 2000, FWD testing was completed for the Western Extension of Highway 407. Initially, FWD testing was completed on the top of the granular base prior to the placement of the HMA. The testing was completed at 100-m intervals in Lanes 1 and 3 in both the eastbound and westbound directions. The testing was initially used to assist in identifying areas of inconsistent subgrade support for remedial action prior to the placement of the granular layers.

A 450-mm diameter loading plate and loads of 15 to 20 kN were used. The photographs in Figures 1 and 2 show the FWD testing on the top of the granular base prior to the placement of the open graded drainage layer (OGDL).

The purpose of the FWD testing was to determine the increased structural support provided by successive pavement layers and to provide layer moduli for use as inputs in a mechanistic check. In addition, the testing is to be used to verify that design assumptions regarding material properties are being achieved in the field. If material properties are exceeded, or designs determined to be "over-designed," it would be possible to determine an economized design based on reduction or elimination of pavement layers or to provide an update of the pavement structural life for input into the pavement maintenance and rehabilitation life-cycle model.

Figure 1. FWD Testing on Top of the Granular Base Layer.

Figure 2. Close-Up of FWD Testing the Top of the Granular Base.

Difficulties with Analyzing FWD Data Collected Over Granular Layers

The case of testing with an FWD over unbound (granular) layers presents several difficulties when analyzed using elastic layer theory-based techniques, due to deviations from elastic theory assumptions. For example:

- Unbound layers do not "ideally" distribute the load horizontally due to a lack of shear strength, forming a discontinuity in the deflection basin.
- The maximum deflection (D0) measured under the plate is artificially increased due to the lack of load distribution, resulting in lower than normal backcalculated moduli.
- Very low deflections occur at distances of 600 mm or more from the load centre, rendering the data basically useless.
- Unbound materials are stress-sensitive, with fine-grained soils being stress softening, while coarse-grained materials are stress hardening. As pavement layers are added, the stress states in lower layers are altered, possibly resulting in the backcalculation of different moduli for the same material.
- FWD-determined moduli are different from laboratory-determined moduli for the same materials, due to inherent differences between the test methods and interpretation of results. Typically, fine-grained subgrade soils tested at low FWD stress levels (akin to "finished pavement" conditions) are 3 to 4 times higher than laboratory determined resilient moduli for the same material (for deviator stresses of 0.4 kPa).

Typically, subgrade resilient moduli are determined using an FWD sensor positioned some distance from the load. However:

- In the commonly accepted stress bulb concept, load is distributed by upper pavement layers, and the surface deflections at a sufficiently far distance from the load plate are considered representative of the subgrade modulus.

- As pavement layers are added, the deflection basins predicted by elastic theory follow the trend seen in Figure 3.

- Figure 4 shows the three average deflection basins for a section tested at completion of the subgrade, base layer 1, and base layer 2.

- Additional layers result in a decrease in D0 (deflection measured under the loading plate), but deflections from D300 (deflection measured at 300-mm offset from the loading plate) outwards do not conform to elastic theory, with the subgrade basin crossing the base layer 1 and base layer 2 basins.

- Deflections from D600 outward are too small to be practically useful.

- Outer deflections are typically used in Boussinesq equations (elastic theory) to determine equivalent subgrade elastic moduli (Es). Deflection and modulus are inversely proportional, with higher deflections producing lower elastic moduli. The backcalculated moduli are referred to as “surface” or “single-layer” moduli (Eo), since the entire pavement is considered to consist of a single layer.

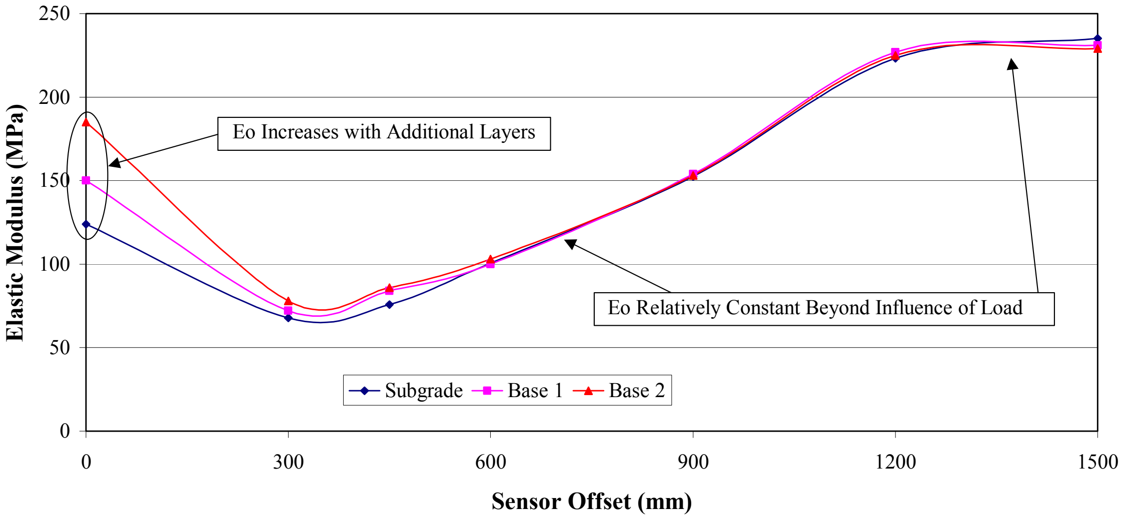

Figure 3. Theoretical Backcalculated Eo versus Sensor Position.

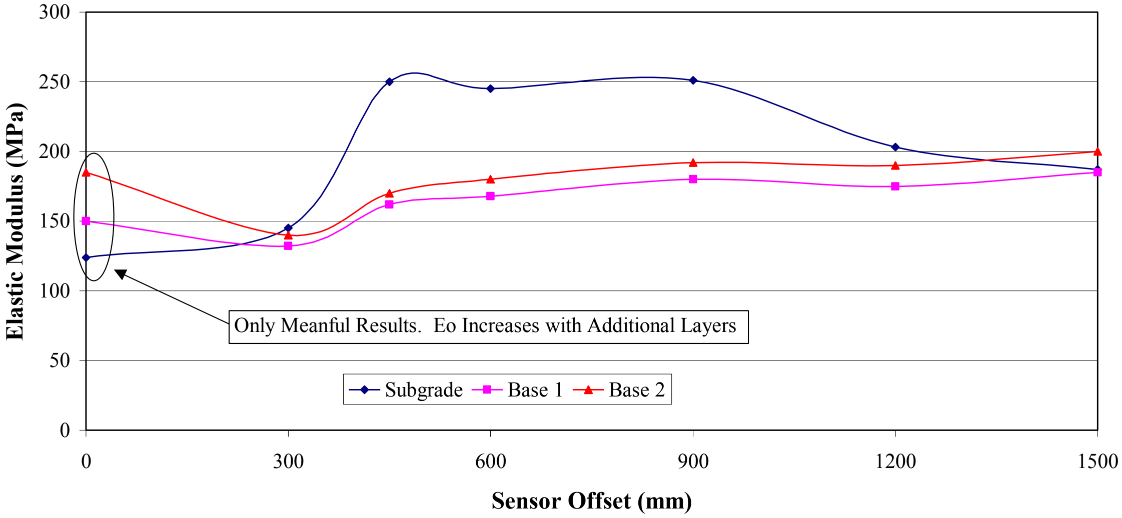

Figure 4. Backcalculated Eo versus Sensor Position.

- For each additional pavement layer, the Eo value at 0 mm increases, due to the increase in stiffness of the layer. At outer sensors the Eo values are constant.

Therefore, it was concluded that the only useful data in this case of deflections measured on top of the subgrade or granular base layers are the deflections at D0, and all other sensor data should be neglected. While it is recognized that D0 values are artificially higher than expected due to compression beneath the load plate, the slight increase in deflection is minor when compared to the total D0 magnitude (i.e., less than 5 percent of the total deflection).

In addition, the results of the testing were quite variable and not considered to be representative. This was partly due to the presence of loose granular particles on the surface. It should be recognized that the FWD sensors are measuring the deflection of the surface of the pavement to an accuracy of 1 µm, with the overall deflection on the order of 500 µm on top of the granular base. Loose particles of granular material under the loading plate would tend to increase the measured deflection. Therefore, the role of the FWD testing was modified from evaluating the structural contribution of each of the pavement layers during construction to confirming overall pavement structural capacity. For this purpose, FWD testing was completed on top of the final lift of binder course HMA.

At each of the test locations, a series of four load applications were applied to the pavement surface. The first application was a "seating" load to ensure the FWD load plate was firmly resting on the pavement surface. The next three loads were approximately 30, 40, and 50 kN. The FWD loading plate and sensor configuration was established to permit the use of closed form mathematical solutions to determine the pavement layer properties in accordance with the 1993 AASHTO *Guide for Design of Pavement Structures* [*2*].

The pavement deflections measured with the FWD at specific distances from the load plate were used to determine the structural properties of the pavement and subgrade through a backcalculation process which uses analytical pavement response models to predict deflections based on a set of given layer thickness and moduli. With pavement thickness held constant, the response models identify the set of subgrade and pavement layer moduli that produce deflections that are very similar to those measured in the field.

The procedure outlined in the AASHTO 1993 *Guide for Design of Pavement Structures*, Part III, Chapter 5 [*2*], was used to determine the properties of the as-constructed pavements. The resultant data include the composite elastic modulus (Ep) for the combination of all bound layers above the subgrade (e.g., the asphalt concrete and granular bases), the subgrade elastic modulus (Es), and the subgrade resilient modulus (M_r).

Based on the backcalculated pavement moduli, the effective structural number (SNeff) of the existing pavement was calculated using the 1993 AASHTO *Guide for Design of Pavement Structures* [*2*] procedure. The effective structural number of the pavement constructed to the upper binder course HMA was calculated. This value was then increased by the expected contribution of the 40 mm of dense friction course (DFC) HMA that has yet to be placed. This value was then subtracted from the design structural number (16.1 cm) to determine the structural deficiency, if any, of the existing pavement.

The results of the FWD analysis for a section the Highway 407 Western Extension are summarized in Table 7

Table 7 Example Summary of FWD Test Results.

Parameter/Direction		**Eastbound**		**Westbound**	
		Lane 1	**Lane 3**	**Lane 1**	**Lane 3**
Dynamic Deflection (mm)	Mean	0.12	0.12	0.11	0.10
	Std. Dev.	0.02	0.02	0.02	0.02
Subgrade Modulus - M_r (MPa)	Mean	88	90	95	104
	Std. Dev.	25	21	28	33
Combined Modulus (MPa)	Mean	1783	1785	1939	2083
	Std. Dev.	271	257	343	296
Structural Number (cm)	Mean	18.3	18.3	18.8	19.3
	Std. Dev.	1.0	0.9	1.1	0.9
Final Structural Number (cm)	Mean	20.1	20.1	20.6	21.0
	Std. Dev.	1.0	0.8	1.1	0.9
Equivalent Asphalt Deficiency (mm)	Mean	None	None	None	None
	Std. Dev.	-	-	-	-

The FWD test results in this section were variable. The normalized dynamic deflection values ranged from about 0.08 to 0.2 mm, with the majority of values on the order of about 0.1 mm. M_r values ranged from about 45 to 150 MPa. The average M_r value was on the order of 90 MPa with a standard deviation of about 25 MPa. While somewhat variable, the M_r exceeded the assumed design value at all test locations. Further, the in-situ structural capacity of the pavement as given by the anticipated final SN exceeded the design value of 16.1 cm in all locations.

The pavement structural design for Highway 407 is based on the AASHTO 1993 *Guide for Design of Pavement Structures*. The design structural number (SN) or structural capacity is determined based on the mean subgrade resilient modulus (M_r) and traffic information. The AASHTO design introduces design safety factors through the concepts of the reliability and standard deviation inputs. The structural capacity of the pavement is provided by the pavement structure layers (asphalt, OGDL and granular base). In the AASHTO design procedure, the individual pavement layers are assigned a structural layer coefficient. The structural layer coefficients are multiplied by the individual layer thickness to determine the structural number (SN) of the pavement. The design SN value for the Highway 407 Western Extension was 16.1 cm.

The FWD test data was analysed and used to calculate the effective structural capacity (SNeff) of the as-constructed pavement. The pavement is considered to be structurally adequate when SNeff is greater than SN. The results of the FWD analysis indicate that this requirement has been met.

The FWD analysis based on the fall 2000 testing indicates that the average M_r is in excess of 80 MPa. Unlike the SN, the M_r is subject to seasonal variations. The subgrade is at is structurally weakest during the spring when moisture conditions are high. It is the strongest in the winter when it is frozen. The AASHTO designs for the Highway 407 pavement were completed using the spring (weakest) M_r value. The estimated M_r value used by for design purposes was 25 MPa. In order to compare the results of the fall FWD testing to the design values, it would be necessary to apply seasonal adjustment factors to the fall values.

Based on previous FWD testing in the Province of Ontario, subgrade modulus seasonal adjustments on the order of 20 to 30 percent would be expected between the spring and fall values. Backcalculation results from Long Term Pavement Performance (LTPP) sites in Manitoba, Minnesota, and Illinois suggested adjustment values on the order of 25 percent. The mechanistic analysis completed to confirm the suitability of the proposed AASHTO pavement designs used a 50 percent reduction in M_r between the fall and spring.

Limited laboratory resilient modulus testing was completed for this project. The testing was conducted at varying deviator stresses and at optimum moisture content and at optimum moisture content plus 2 percent. Based on a deviator stress of 21 kPa, the reduction in M_r ranged from 45 to 65 percent.

Based on FWD seasonal variation testing and the limited laboratory resilient modulus testing completed, it was expected that the spring M_r values will be on the order of 25 to 50 percent lower than the fall values. In order to establish site-specific seasonal adjustment factors for M_r, spring FWD correlation testing was completed in 2001.

The results of the spring FWD testing are presented in Table 8

Table 8 Results of the Spring 2001 FWD Test Results.

Parameter/Direction		Eastbound		Westbound	
		Lane 1	Lane 3	Lane 1	Lane 3
Dynamic Deflection (mm)	Mean	0.16	0.14	0.14	0.13
	Std. Dev.	0.04	0.05	0.03	0.05
Subgrade Modulus - M_r (MPa)	Mean	70	78	80	86
	Std. Dev.	19	17	14	14
Combined Modulus (MPa)	Mean	1475	1560	1598	1683
	Std. Dev.	291	243	197	276
Structural Number (cm)	Mean	17.2	17.5	17.5	17.9
	Std. Dev.	2.1	2.4	1.1	1.1
Final Structural Number (cm)	Mean	18.9	19.2	19.2	19.6
	Std. Dev.	2.1	2.4	1.1	1.1
Equivalent Asphalt Deficiency (mm)	Mean	None	None	None	None
	Std. Dev.	-	-	-	-

Note: Data summary for the entire Highway 407 Western Extension Section

A comparison of the fall and spring measured subgrade modulus values are presented in Table 9

Table 9 Comparison of Fall and Spring Subgrade Modulus Values.

Section	Mean Subgrade Modulus (MPa)		Percent Difference
	Fall 2000	Spring 2001	
Eastbound Lane 1	88	70	-20
Eastbound Lane 3	90	78	-12
Westbound Lane 1	95	80	-16
Westbound Lane 3	104	86	-17
Western Extension Average	94	79	-16

The comparison of the mean subgrade modulus values between the fall of 2000 and the spring of 2001 indicated a reduction of about 15 to 20 percent which is somewhat less than that estimated based on the laboratory testing.

CONCLUSIONS

The design, analysis and design verification procedures outlined in this paper demonstrate the usefulness of mechanistic pavement design procedures in developing suitable pavement designs for a design/build project. Specific conclusions that can be drawn from this analysis include:

- Mechanistic pavement design checks can be used effectively for pavement maintenance and rehabilitation planning.

- The analysis of Falling Weight Deflectometer testing completed on the top of the subgrade and granular base layers can be problematic at best, as unbound layers do not distribute the applied load horizontally due to the lack of shear strength in the material.

- Very low surface deflections beyond the loading plate may result in an over-estimation of the subgrade modulus, which could lead to unconservative designs.

- FWD-determined subgrade modulus values are different from laboratory-determined moduli for the same materials due to inherent difference in the test methods and stress state.

- The low standard deviation of the FWD test results, particularly with respect to the effective subgrade modulus suggests a high quality of uniformity of pavement construction.

- The FWD is a useful tool to assist in validating mechanistic pavement designs.

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