

**Impact of Road Geometry and Thickness on Pavement Behaviour using PSIPave 3D™**

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## ABSTRACT

Roads allow for the transportation of people and goods, thus fostering economic growth and acting as an integral piece of infrastructure in any community. The design of the pavement structure and geometry of a road has a contribution to the long-term performance of the road. The primary objective is to distribute the stresses from traffic on the surface to the layers below and ultimately the in-situ subgrade. By properly selecting designs and planning strategies, the deterioration rate can be decreased, saving costs for maintenance and repairs and minimizing disruption to users while providing a better performing asset. Many current pavement design methods use empirical predictions and do not truly account for material behaviour under loads. Conventional pavement design methods such as American Association of State Highway and Transportation Officials 93 (AASHTO 93) and Shell Design Charts for Flexible Pavements (Shell) express traffic in the form of equivalent single axle loads which may not accurately reflect the various truck loading configurations to which a pavement is subjected. This research is done in collaboration with PSI Technologies Inc. (PSI) and the City of Hamilton to compare conventional pavement design methods and the effect of road geometry on pavement structure behaviour. PSI has developed a three-dimensional finite element pavement design method, PSIPave 3D™. This method is a mechanistic model that fully describes the pavement structure behaviour considering traffic loading, roadway geometry and material properties. AASHTO 93, Shell, and Mechanistic-Empirical Pavement Design Guide (MEPDG) do not fully consider the impact of road geometry and material behaviour. This project considers a flexible, urban collector road section in Hamilton, ON. Twelve cases of pavement structure reconstruction and roadway geometry have been developed. Using PSIPave 3D™, the designs and performance predictions for each case are discussed in this work.

KEYWORDS: Pavement design methodologies, mechanistic-empirical pavement design guide (MEPDG), AASHTO 93, Shell, pavement performance, PSIPave 3D™

## 1 INTRODUCTION

The function of a pavement structure is to smoothly and safely carry traffic loads. The pavement designer must ensure that the pavement structure is structurally adequate and provides a safe surface for vehicle travel, while maintaining rideability. Advancements in pavement design have been limited over the last several decades. Agencies continue to use known practices such as American Association of State Highway and Transportation Officials 93 (AASHTO 93) and Shell as these methods are familiar and require limited data to run. The outputs are well understood and easily implemented. In the example of AASHTO 93, given its empirical basis and development from a specific set of data, it has been identified to have the potential to produce pavement structures that are conservative or not entirely representative of the project conditions [1]. An empirical approach cannot fully account for material behaviour under traffic loading and road geometry. This work looks at conventional pavement design methods, such as AASHTO 93, Shell and Mechanistic-Empirical Pavement Design Guide (MEPDG), and a finite element model, PSIPave 3D™.

### 1.1 BACKGROUND

The pavement design methods commonly used in Canada are generally empirical or mechanistic-empirical. Empirical methods are based on experience or experiments while mechanistic-empirical methods relate the stresses, strains and deflections to the applied loads through empirical engineering equations [2]. Both design methodologies have been used successfully in the past, but have limitations and do not consider roadway geometry, climatic conditions and the true behaviour of the materials with each having a contribution to pavement performance [3].

Because of their common use in Canada, the pavement design methods considered in this paper are AASHTO 93, MEPDG and Shell. Table 1 presents the pavement design methods typically used in provinces and territories across Canada based on the Table outlined in the 2013 Pavement Asset Design Management Guide by the

Transportation Association of Canada. The recommendations of the three design tools will be compared using PSIPave 3D™. The three design methods and PSIPave 3D™ are briefly described in the following section.

Table 1. Flexible Pavement Design Methods in Canadian Provinces [4]

Agency	General Design Methods
Alberta	AASHTO 93
British Columbia	AASHTO 93
Saskatchewan	Shell Method Asphalt Institute
Manitoba	AASHTO 93/MEPDG (new construction) Canada Good Roads Association (CGRA)/MEPDG (rehabilitation)
Ontario	AASHTO 93 Ontario Pavement Analysis of Costs (OPAC) Routine (Empirical) Method
Quebec	AASHTO 93 Chausée 2
New Brunswick	AASHTO 93 Rebound Values
Prince Edward Island	Asphalt Institute
Nova Scotia	AASHTO 93 Correlation Charts using AADT and grain size of subgrade
Newfoundland and Labrador	Standard Section Used
Yukon	State of Alaska Design Method
Public Works and Government Services Canada (Federal)	AASHTO 93 State of Alaska Design Method

In this work, the design methods are being considered and discussed regarding flexible pavement design.

### 1.1.1 AASHTO 93

The pavement design method, AASHTO 93, is empirical and was developed by The American Association of State Highway and Transportation Officials. This method is based on the results of a 2-year road test in Ottawa, Illinois. The AASHTO 93 empirical equation differentiates pavement materials with layer coefficients and describes traffic with Equivalent Single Axle Loads (ESAL).

A benefit of AASHTO 93 is that it requires a limited number of inputs to run. This allows it to be used in a variety of scenarios. It has been successfully used in the past and is still being used in many provinces and agencies given its simplicity and familiarity. Since the AASHTO 93 equation is based on experimental data, the resulting equation only considers specific loads, geometric design and climatic conditions encountered in the two-year evaluation period [2]. Therefore, it does not fully account for material behaviour under loading, geometric and climatic changes [3]. Typically, AASHTO 93 does not differentiate between material changes. For example, all layers of new asphalt within a pavement structure are generally described with the same layer coefficient.

### 1.1.2 Shell

Shell is a mechanistic empirical design method developed by Shell International Petroleum Company. This method is based on the linear elastic multi-layer system which helps obtain the maximum tensile strain at the bottom asphalt layer and the maximum compressive strain at the top of the subgrade [5]. This method requires traffic data, dynamic modulus of each layer, mix code, and the mean annual air temperature. Shell also utilizes the number of 80 kN single axle loads to describe traffic (ESALs). The mix code assigns the properties of the

asphalt mix such as its penetration resistance, fatigue, and stiffness. Once these inputs are available, the designer can refer to a set of curves to determine the thickness of the asphalt layer and unbound base and subbase layers. Shell only considers climate through the mean annual air temperature which does not account for all effects of climate. Shell does not consider the geometry of the road.

### **1.1.3 MEPDG**

Mechanistic-Empirical Pavement Design Guide (MEPDG) is a mechanistic empirical design method. AASHTOWare Pavement ME Design is a software that enhances the use of MEPDG to find stresses, strains, and deflections to determine the distresses throughout the pavement's design life. Pavements can be analyzed in MEPDG at three levels. Level one is intended to provide the highest accuracy as it requires the largest number of inputs, which describe the traffic and materials in detail but can require significant resources to obtain. In comparison, level three uses more typical values for a given region and assumed materials. However, even typical values need to have been developed.

MEPDG calculates the terminal IRI, permanent deformation (asphalt and entire pavement structure), thermal cracking, and bottom-up fatigue cracking for a specific scenario. The distress results are compared to the required performance criteria for the region to determine whether the pavement design in consideration is satisfactory. The distress prediction model is still under development for some distresses, such as top-down cracking [6].

MEPDG includes inputs such as climate conditions, material properties, and traffic which each have a role in pavement performance, but road geometry is not considered. In order to achieve accurate results and predictions, the performance equations must be calibrated and verified. Local calibration improves accuracy as it transfers the global performance equations to equations that are more fit for the region, reducing the standard error between measured and predicted distresses [7]. Minimizing the development of surface distresses is one of the goals of pavement design and therefore MEPDG can be a powerful tool once it is locally calibrated and material parameters are known. The completion of these two efforts can be resource intensive and unattainable for many agencies. MEPDG is not a traditional pavement design method as it does not produce a suggested pavement structure thickness. Instead it evaluates the pavement structure that is provided by the designer.

### **1.1.4 PSIPave 3D™**

PSIPave 3D™ is a mechanistic design method based on three-dimensional finite element modelling to determine the stresses and strains within a pavement structure. Therefore, critical normal strains, shear strains, and deflections can be obtained and used to identify failure limits. PSIPave 3D™ requires the user to provide information regarding the road geometry, layer thicknesses, material properties, and load configuration.

PSIPave 3D™ is an analysis tool that provides insight into areas that most other methods do not account for such as the impacts of road geometry, tire pressure, and truck configuration. The other pavement design tools included in this paper are commonly used in Canada and do not have the capacity to provide the user with information about the shear stresses generated from vehicle loading on the pavement structure. Stresses are generated from braking, acceleration and standard motion of the wheel that occur while driving [8]. Research has shown that major distresses in pavements are a result of many factors, including shear stresses which are presented in 3D finite element modelling [9]. Shear strains are only quantified through three-dimensional modelling which is utilized by PSIPave 3D™ to address one of the limitations of conventional methods. This method shows the significance of loading configuration, road geometry and material properties on the pavement structure performance through the normal strains and deflection.

## 1.2 LITERATURE REVIEW

Previous studies have been done on conventional pavement design methods that highlight their differences and limitations. A study in Manitoba showed that increasing the subbase by 200 mm increased the pavement life by 0.25 years. In AASHTO 93, this same change in the subbase thickness would double the pavement life. This study also looked at the difference in using local axle load spectra versus MEPDG default values. Their findings were that the MEPDG default values were not representative of local conditions [10]. This shows the importance of local calibration.

Another study aimed at the comparison of AASHTO 93 and MEPDG has shown that MEPDG produces thinner designs for asphalt highways in Kansas, United States [11]. Aguib also identified the tendency of AASHTO 93 to produce thick pavement structures in Egyptian cities considering low, medium and high traffic levels. They acknowledged the importance of capturing traffic loading and environmental conditions in the inputs of pavement performance prediction [12]. These results show that similar findings are shown outside of North America as well.

It has been shown that a 5% increase in temperature can reduce the pavement service life by over 20% [13]. A 1% increase in moisture can reduce the resilient modulus of the soil by 30% [14], therefore weakening the entire pavement structure. This highlights the importance of environmental effects on the pavement structure and the limitations of empirical tools. Therefore, conventional pavement design methods do not consider all the contributing factors to pavement performance which have the capability to severely damage the pavement.

## 1.3 OBJECTIVES

The objective of this work is to develop a new pavement design for North Service Road in the City of Hamilton (the City) using AASHTO 93, Shell and MEPDG then use PSIPave 3D™ to further evaluate the behaviour of the materials in each pavement structure design and the impact of road geometry on these designs.

## 2 RESEARCH METHODOLOGY

The University of Waterloo (UW) and PSI have collaborated with the City to evaluate multiple road sections using data collected by PSI's survey van, provide pavement rehabilitation alternatives for each road section and evaluate the alternatives in PSIPave 3D™. This paper uses one of the road sections to compare the reconstruction recommendations of three design tools (MEPDG, Shell and AASHTO 93) and evaluate the pavement structure behaviour of each using PSIPave 3D™. In addition to the Ground Penetrating Radar (GPR), LIDAR and rutting data collected with PSI's van the City has provided borehole logs and traffic data.

The pavement design methodologies considered in this work do not all provide the same outputs nor need the same inputs. AASHTO 93 and Shell both provide the designer with a means of determining the pavement structure layer thicknesses. Meanwhile, MEPDG requires layer thicknesses as an input to predict the distresses. PSIPave 3D™ outputs the stresses, strains, and deflection anywhere along the structure, but most importantly the critical normal strains, shear strains and deflections.

The most efficient way to compare these methods will be to convert all outputs into thicknesses so that they can be analyzed in PSIPave 3D™. For MEPDG, layer thickness is one of the inputs and since MEPDG is an iterative design process, the thicknesses can be modified until the design satisfies the required reliability. A minimum threshold for the thickness will be set for the MEPDG design based on the City's minimum pavement design structure for the given road classification.

Twelve cases have been developed for this research, using the collected field data. Three different design tools, AASHTO 93 (Case A), Shell (Case B) and MEPDG (Case C) were used to design the pavement thicknesses. Within each thickness, the following four geometric cases were considered and run using PSIPave 3D™:

- Case 1: narrow shoulder with a steep side slope
- Case 2: narrow shoulder with a gradual side slope
- Case 3: wide shoulder with a steep side slope
- Case 4: wide shoulder with a gradual side slope

### 3 ROAD SECTION

North Service Road (NSR) is a two-lane urban collector located in Hamilton, ON and is approximately 5.35 centerline kilometers in length as shown in Figure 1. This section starts at Fruitland Rd. and ends at Centennial Pkwy. The cross section of the road varies from rural with ditches to a combination of rural and urban with curbs and catch basins.

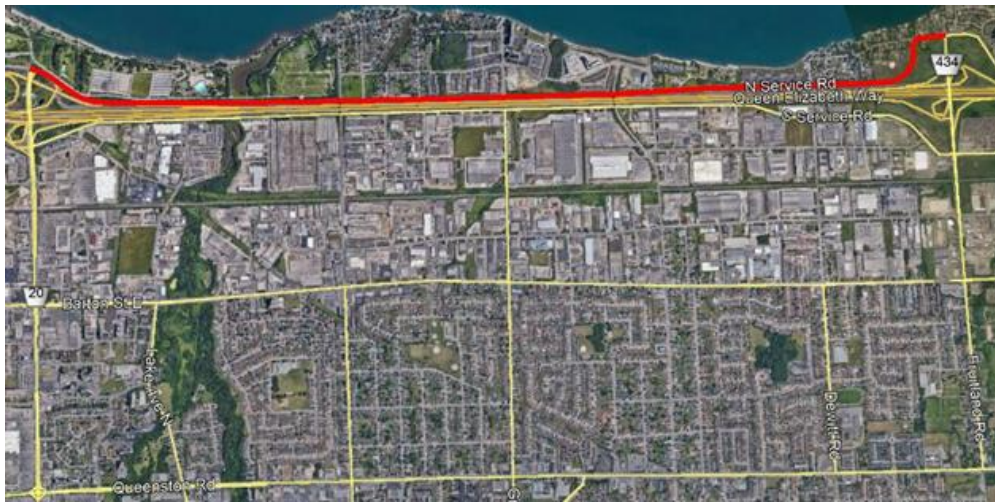


Figure 1. North Service Road [15]

In the City’s Comprehensive Development Guidelines and Financial Policies Manual, minimum pavement structure layer thicknesses for all road classifications [16]. For urban collectors, the specified minimum design is:

- 40 mm of SP9.5PG58-28
- 80 mm of SP19 PG58-28
- 150 mm of Granular A
- 300 mm of Granular B type II.

### 4 DESIGN

A 20-year reconstruction pavement design was developed using each of the three methods: AASHTO 93, Shell and MEPDG. The data sources, inputs and designs are provided in this section.

#### 4.1 DATA

In addition to the data collected on North Service Road using PSI’s survey van, a set of reports was provided by the City. These reports contained data such as borehole/core logs, traffic counts, and sieve charts.

##### 4.1.1 Borehole Logs

All design methods considered in this work required information about the existing subgrade material. Available borehole data was provided by the City. The logs identified that the material underlying the existing granular

pavement structure layers mostly consisted of silty clay (CL). This silty clay material was assumed for the entirety of the subgrade, including the subgrade side slope.

The “AASHTO Pavement Design Guide for Ontario Conditions” specifies value ranges for the resilient modulus of different soil types [17]. A silty clay has a resilient modulus ranging from 15 MPa to 35 MPa, considering the condition of the material from poor to good. An average value of 25 MPa was chosen as further information, aside from a few gradation results was not available for the material. Laboratory testing can be costly but the more that is understood about the existing materials and materials used in the design, the more suitable a design can be.

The provided gradation results were used to assess the frost susceptibility of the material using the frost susceptibility criteria [18]. The silty clay material was identified to be highly frost susceptible. The designs should consider the frost susceptibility of the underlying subgrade material and plan accordingly for thickness and drainage. In the City the frost depth is 1.2 m [19].

#### 4.1.2 Traffic Data

Traffic counts, including trucks and buses, on an interval of 15 minutes, were provided by the City for North Service Road. The ESALs were estimated with the following parameters to be 4.5 million for a 20-year design period:

- AADT - 7640
- Heavy Vehicle Percentage (HVP) – 4%
- Truck Factor – 2 (FHWA 4), 0.3 (FHWA 5), 0.9 (FHWA 6), 4 (FHWA 7), 1.1 (FHWA 8), 1.6 (FHWA 9), 4 (FHWA 10), 1 (FHWA 11), 4.3 (FHWA 12), 5.6 (FHWA 13)
- Lane distribution – 1
- Directional Factor – 1
- Growth Factor – 24.3 based on a growth rate of 2% [20]
- Number of days in a year – 365

#### 4.1.3 Road Geometry

Based on the LIDAR data obtained from the PSI survey van, a summary for the road geometry is shown below. These values are on the north side of North Service Road where steep slopes and narrow shoulder widths occur:

- 25.9% of the shoulder widths are between 0 to 1.5 m, 59.3% are between 1.5 to 3 m, and 14.8% are over 3 m.
- The north side slope average is 1:3.8.

### 4.2 AASHTO 93 DESIGN

Using the “AASHTO Pavement Design Guide for Ontario Conditions”, the following inputs were used for a reconstruction design of North Service Road [17]:

- ESALs – 4,500,000
- $S_o$  – 0.49
- Reliability – 90 %
- $\Delta$ PSI – 2.2
- $M_r$  – 25 MPa

AASHTO 93’s empirical equation generated a Structural Number (SN) of 135. The SN provides a minimum value that the pavement structure must meet using Equation 1

$$SN^* = \sum_{i=1}^n m_i \times d_i \times a_i \quad \text{Equation 1}$$

Where:

SN\* is Structural number of design (mm)

n is Number of layers

$m_i$  is Drainage coefficient of layer i

$d_i$  is Thickness of layer i (mm)

$a_i$  is Layer coefficient of layer i

The design structural number calculated with Equation 1 must be greater than the required structural number of 135. Layer coefficients used were 0.44, 0.14 and 0.12 for Asphalt, Granular A, and Granular B, respectively [17]. The drainage coefficient for new designs is 1.

Carrying out an overstressing check, it was determined that the minimum asphalt layer thickness is required to be 131.8 mm. From here, it is up to the designer to come up with thicknesses that are reasonable, considering constructability, City guidelines, experience and frost susceptibility. The following pavement structure was determined:

- 140 mm of Asphalt
- 150 mm of Granular A
- 450 mm of Granular B Type II

In Ontario, the typical thickness of Granular A used is 150 mm. From there, the thickness of Granular B was increased until the design structural number of the whole pavement structure was higher than 135. This design aimed for higher than 700 mm to provide extra protection against frost as the subgrade was heavily susceptible to frost. This design meets the City of Hamilton's minimum pavement structures.

### **4.3 SHELL DESIGN**

The required inputs in Shell are: Mean Annual Air Temperature (MAAT); layer moduli; ESALs; and mix code.

#### **4.3.1 MAAT**

The MAAT value for Hamilton is 13.2°C. A MAAT value of 12°C was selected for the design based on available Shell curves.

#### **4.3.2 Layer Moduli**

A subgrade modulus of 25 MPa was used for the design. The Shell pavement design manual includes a chart which specifies the dynamic modulus for each major soil classification. Based on this chart, the moduli chosen for Granular A and Granular B is 800 MPa and 400 MPa, respectively [5].

#### **4.3.3 Mix Code**

A mix code must be determined for the asphalt that is intended to be used. There are three steps to identifying the right mix code. Stiffness described by S1 and S2, fatigue described by F1 and F2, and the penetration value of 50 or 100. S1/F1 accounts for mixes with average properties (bitumen, aggregate, and void size), and S2/F2 is for open-graded mixes [5]. Ontario and the City has used the Performance Graded (PG) asphalt cement referencing system for several years however penetration grade is still used in many parts of the country. For penetration resistance, 100 was selected as it was identified to be more reflective of the PG grades used in the City, than 50. For this design, a mix code of S1-F1-100 was used for North Service Road.

The inputs for the Shell design are the following:

- Traffic: 4.5 million ESALs
- MAAT: 12°C
- Mix code: S1-F1-100



In Shell design, the thickness of the unbound layers is initially chosen within the subgrade modulus requirements. A thickness of 600 mm was chosen for this design for the unbound granular layers. The final Shell design is shown below.

- 135 mm of Asphalt
- 260 mm of Granular A
- 340 mm of Granular B Type II

Once the total thickness of unbound granular layers is chosen, Shell determines the breakdown of the base and subbase layers based on their dynamic moduli to achieve the required structural capacity, as such 150 mm was not used for layer thickness of Granular A.

#### 4.4 MEPDG DESIGN

In the MEPDG design, the AASHTOWare software has been used. A Level one (site-specific) design was carried out where possible however Level 3 inputs were required for some items and were obtained from “Ontario’s Default Parameters for AASHTOWare Pavement ME Design Interim Report – 2019” [21]. Ontario uses the following performance threshold parameters for an MEPDG design:

- Initial International Roughness Index (IRI): 1 (new asphalt concrete)
- Terminal IRI: 2.7 (collector)
- Top-down fatigue cracking: 380 m/km
- Bottom-up fatigue cracking: 35% (collector)
- Total permanent deformation: 17 mm (collector)
- AC thermal fracture: 190 m/km
- Total cracking: 50%

The materials used for the MEPDG design are shown in Table 2 and explained below.

Table 2. North Service Road Layer Material Types

Layer	Materials
Surface	SP 12.5 PG 70-28
Binder	SP 19 PG 64-28
Base	Granular A
Subbase	Granular B Type II
Subgrade	Silty Clay (A-6)

Level 3 data for PG 70-28 and PG 64-28 of SP 12.5 and SP 19 was used for the dynamic modulus values. The grade used in this design is equivalent or exceeds the anticipated performance of the mix typically used by the City. The traffic data was provided by the City and included an Average Annual Daily Truck Traffic (AADTT) of 310 per lane per direction. An urban collector requires a reliability of 50% for IRI and 80% for the other distress types as per the Ontario default parameters [21]. The minimum threshold for MEPDG was set based on the City’s minimum pavement structure for the given road classification mentioned earlier in the paper.

The City’s recommended pavement structure for a collector with the following thicknesses had performance prediction results that were satisfactory as shown in Table 3. The City’s recommended asphalt thickness was slightly altered as the minimum thickness for each layer in MEPDG was 50 mm.

- 50 mm of SP 12.5 FC2 PG 70-28 (City recommends 40 mm)
- 70 mm of SP 19 PG 64-28 (City recommends 80 mm)
- 150 mm of Granular A

- 300 mm of Granular B Type II

Table 3. AASHTOWare Outputs for MEPDG Optimized Thicknesses

Distress Type	Distress @ Specified Reliability		Reliability		Criterion Satisfied?
	Target	Predicted	Target	Predicted	
Terminal IRI (m/km)	2.7	1.73	50	97.82	Pass
Permanent deformation - total pavement (mm)	17	5.59	80	100	Pass
AC bottom-up fatigue cracking (percent)	35	1.01	80	100	Pass
AC thermal cracking (m/km)	190	27.29	80	100	Pass
Permanent deformation - AC only (mm)	6	0.76	80	100	Pass

Top-down cracking is neglected from the distress types because its failure mechanism is recognized to not be fully understood in the industry [6]. As such, the performance model in MEPDG for top-down cracking is still undergoing development.

The identified pavement structure is 570 mm in thickness, consisting of 48 % of the frost depth. Given the frost susceptibility of the existing subgrade material, a thicker pavement structure may be considered for the final design. However, for the purposes of the comparison in this work, this structure will be used.

#### 4.5 PSIPAVE 3D™

PSIPave 3D™ requires several inputs to complete the finite element analysis, however the required inputs are generally available, including traffic loading information, road geometry and material properties. PSIPave 3D™ also has a materials database which provides a list of typical materials for a specific agency. The outputs from this method provide a great deal of information about the pavement performance and behaviour that is not available from the three other design tools. It provides the user with stress and strain contours at any point within the pavement structure. It also provides the stresses, strains, and deflection under each tire of the identified loading. The higher the number of nodes for the analysis, the greater the detail of the outputs. The outputs provide the user with the maximum normal strains, shear strains, and deflections.

##### 4.5.1 Road Geometry

The road geometry data that is used in PSIPave 3D™ includes: road cross section (ditches or curbs); road width; side slope; surface slope; subgrade slope; shoulder width; and number of lanes. The existing geometry of North Service Road fluctuates, with varying shoulder width and side slopes. Considering the in-situ conditions, the following four road geometries were used in this analysis to compare the impact on pavement performance:

- Case 1: narrow shoulder (0.7 m) and steep side slope (1:1.7)
- Case 2: narrow shoulder (0.7 m) with a gradual side slope (1:6.4)
- Case 3: wide shoulder (3.95 m) with a steep side slope (1:1.7)
- Case 4: wide shoulder (3.95 m) with a gradual side slope (1:6.4)

The other road geometry inputs remained consistent for all four cases and reflect the in-situ conditions:

- Number of lanes – 2
- Lane width – 3.5 m
- Ditch depth – 1 m
- Cross section – rural with ditches for drainage

An example of road geometry inputs in PSIPave 3D™ is shown in Figure 2.

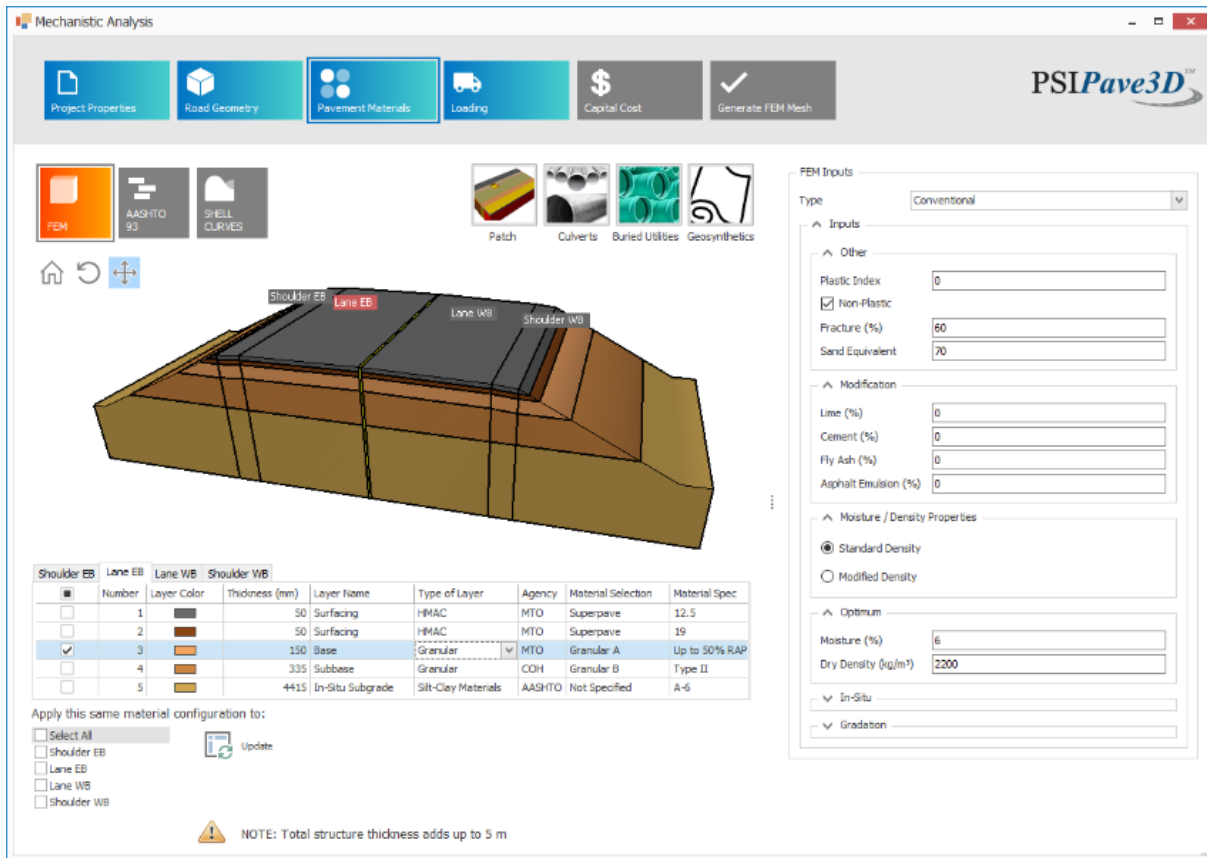


Figure 2. Geometry of Case A for North Service Road

#### 4.5.2 Material Properties

The modulus of each material is required in PSIPave 3D™. Level 1 data for PG 70-28P and PG64-28P of SP 12.5 and SP19 was used for the dynamic modulus values. The P indicates that the asphalt mix is polymer modified. Polymers increase the rutting resistance, fatigue resistance, and thermal cracking resistance of the mix [22]. Dynamic modulus values from previous research carried out at CPATT were used. The Poisson's Ratios and modulus values of the other materials were [21]:

- SP12.5 FC28 PG70-28P (Surface Layer), Modulus: 6934 MPa, Poisson's Ratio: 0.35
- SP19 PG64-28P (Binder Layer), Modulus: 11089 MPa, Poisson's Ratio: 0.35
- Granular A (Base), Modulus: 250 MPa, Poisson's Ratio: 0.35
- Granular B (Subbase), Modulus: 200 MPa, Poisson's Ratio: 0.35
- Silty Clay (Subgrade), Modulus: 25 MPa, Poisson's Ratio: 0.45

#### 4.5.3 Pavement Structure Layers

The pavement structures developed with AASHTO 93, Shell and MEPDG in this work were used in PSIPave 3D™. Each pavement structure was used for four cases, each case with a different road geometry, as described in Section 4.5.1.

#### 4.5.4 Loading

PSIPave 3D™ allows the user to define the vehicle loading, including configuration, axles weights and tire inflation. The vehicle loading in PSIPave 3D™ is chosen based on the worst case scenario. In this work a truck categorized as FHWA 13 with seven or more axles was used which has the highest average ESALS per truck and therefore has adverse effects on the pavement [23]. In Ontario, an example of this truck is a tractor b-train double trailer as shown in Figure 3.

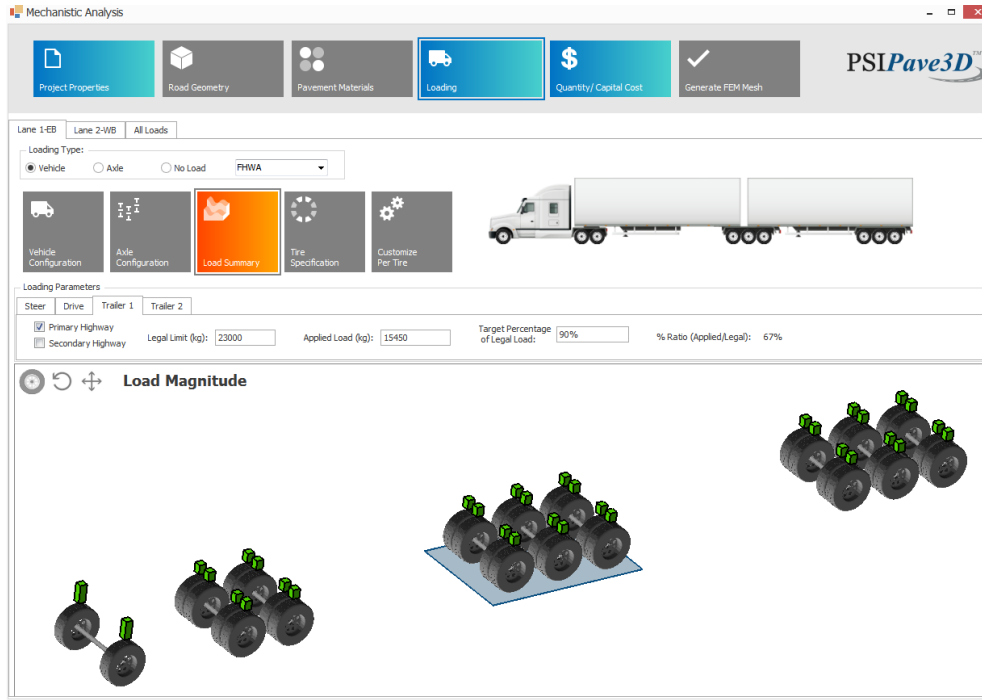


Figure 3. Tractor B-Train Double Trailers in PSIPave 3D

## 5 RESULTS AND DISCUSSION

The results of each pavement design tool are presented in this section and discussed.

### 5.1 PAVEMENT DESIGN THICKNESSES

The pavement structure layer thicknesses developed with each pavement design method are presented in Table 4.

Table 4. Thicknesses Given by Each Design Method

Layer Materials	Pavement Design Thicknesses (mm)		
	AASHTO 93 (Case A)	Shell (Case B)	MEPDG (Case C)
SP12.5 FC2 PG70-28P	60	60	50
SP19 PG64-28P	80	75	70
Granular A	150	260	150
Granular B Type II	450	340	300
<b>Total</b>	<b>740</b>	<b>735</b>	<b>570</b>

Table 4 compares the pavement layer and structure thicknesses developed with each pavement design tool. AASHTO 93 (Case A) generated the thickest total pavement structure of the three design tools, only slightly thicker than the Shell method though. All three methods allow the user to adjust layer thicknesses while still achieving the identified requirement. Frost protection was not considered in the MEPDG case as a failure criterion. If the MEPDG design thickness was changed to meet the frost requirement, the structure would be thicker. However, MEPDG deemed the city’s minimum pavement structure to be satisfactory in all distress criteria and this structure will be used for comparison.

## 5.2 PSIPAVE 3D™ RESULTS

The results generated by PSIPave 3D™ for all 12 cases are presented in the following sections.

### 5.2.1 PSIPAVE 3D™ NORMAL STRAINS

The peak tensile strain in the direction of traffic in the asphalt binder layer expresses the strain caused by traffic which leads to fatigue failure (bottom-up cracking). This type of failure initiates in the bottom of the deepest asphalt layer and propagates upwards. The peak tensile strain measured in the asphalt binder for each case is shown in Figure 4.

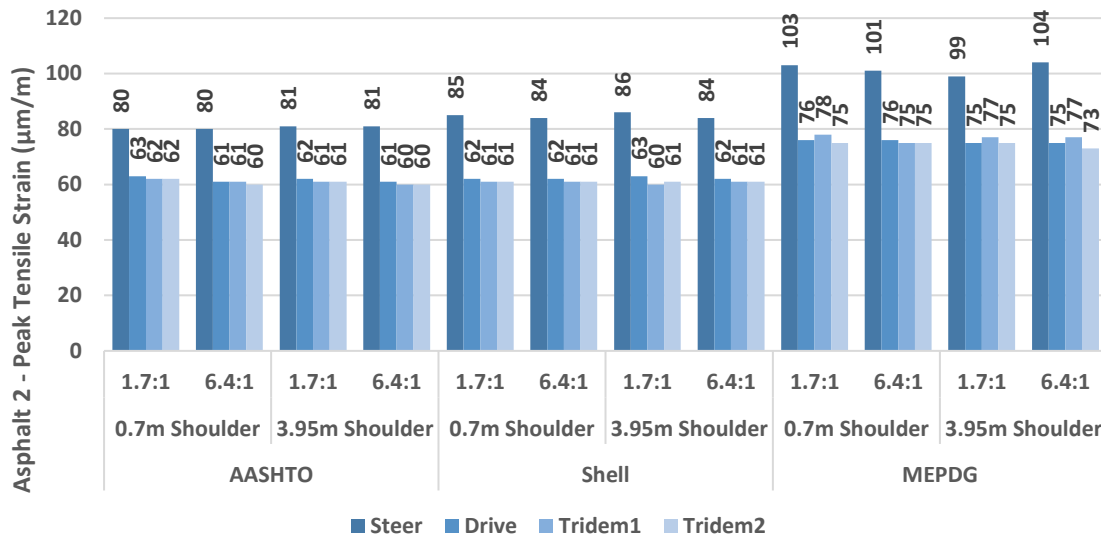


Figure 4. Peak Tensile Strain in Asphalt Binder Layer (µm/m)

The following trends are observed within the peak tensile strain results:

- The tensile strains observed in the MEPDG design are on average 24% more than AASHTO 93 and 22% more than Shell. This is likely because of the thickness difference in the asphalt layers and entire pavement structure. The MEPDG design has only 86 % of the asphalt layer thickness of AASHTO 93 and 89 % of Shell. The asphalt difference in thickness is 5 mm between AASHTO 93 and Shell and 20 mm between AASHTO 93 and MEPDG.
- Overall, the geometry has minimal effects on the tensile strains, though it seems to have more of an impact in MEPDG where the total pavement thickness is lower.
- The area under the steer axle experienced the greatest tensile strain in each case. This is due to the steer axle having a single axle and single tires. The other axles (tandem and tridem) have dual tires which distribute the strains across a larger footprint.

The peak compressive strain in the subgrade is shown in Figure 5. This value describes the tendency for rutting in the subgrade due to loading. When the subgrade deforms, the whole pavement structure sitting on the subgrade also settles and leads to poor performance overall.

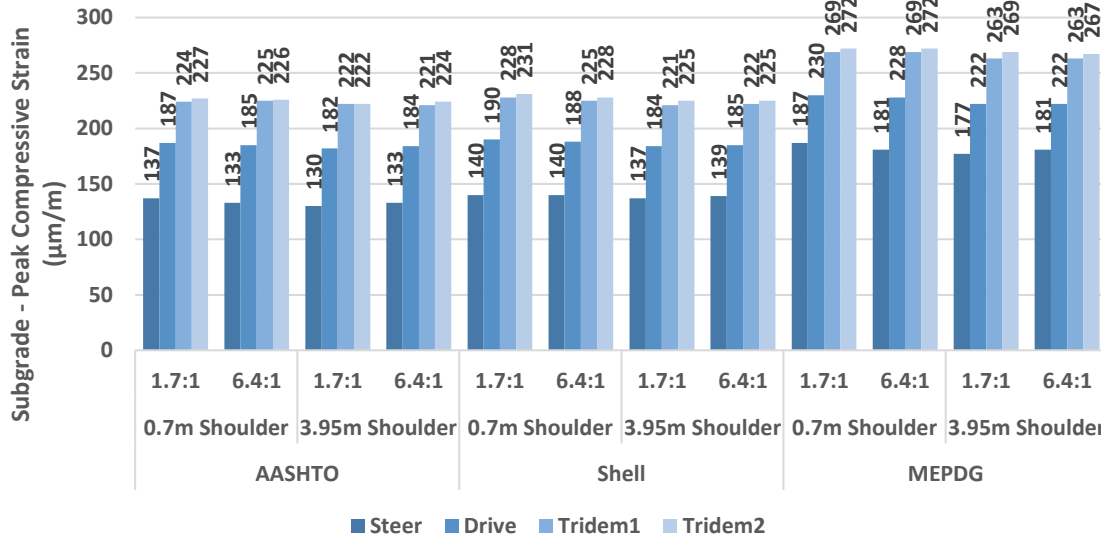


Figure 5. Peak Compressive Strain in Subgrade ( $\mu\text{m/m}$ )

The following trends are observed for the peak compressive strain:

- The compressive strains observed in MEPDG are on average 23% more than AASHTO 93 and 21% more than Shell.
  - The AASHTO 93 design is only 5 mm thicker than the Shell design, and this difference is in the asphalt layer.
  - The distribution of granular material layer thicknesses differs between the AASHTO 93 and Shell but both the base and subbase granular are similar quality material.
  - Shell has a thicker base granular layer which has slightly better properties than the subbase.
- The effects of geometry were minimal on the peak compressive strains though it seems to have more of an impact in MEPDG where the pavement structure is thinner.
  - Even though the effect is minimal, the peak compressive strains are slightly reduced as the transition is made from narrow to wide shoulders.

### 5.2.2 PSIPAVE 3D™ SHEAR STRAINS

Shear strain is a common mechanism of failure of pavement structures, however this parameter is not frequently analyzed due to the requirement to utilize three-dimensional modeling to accurately quantify the behaviour. Given the capability of PSIPave 3D™ to provide this information, the peak shear strains in the asphalt layer, subbase layer, subgrade and subgrade side slope are shown in Figure 6 to Figure 11. Shear distributions and peaks across all layers and lanes were measured, and these figures present a few examples of the results (peak shear strains).

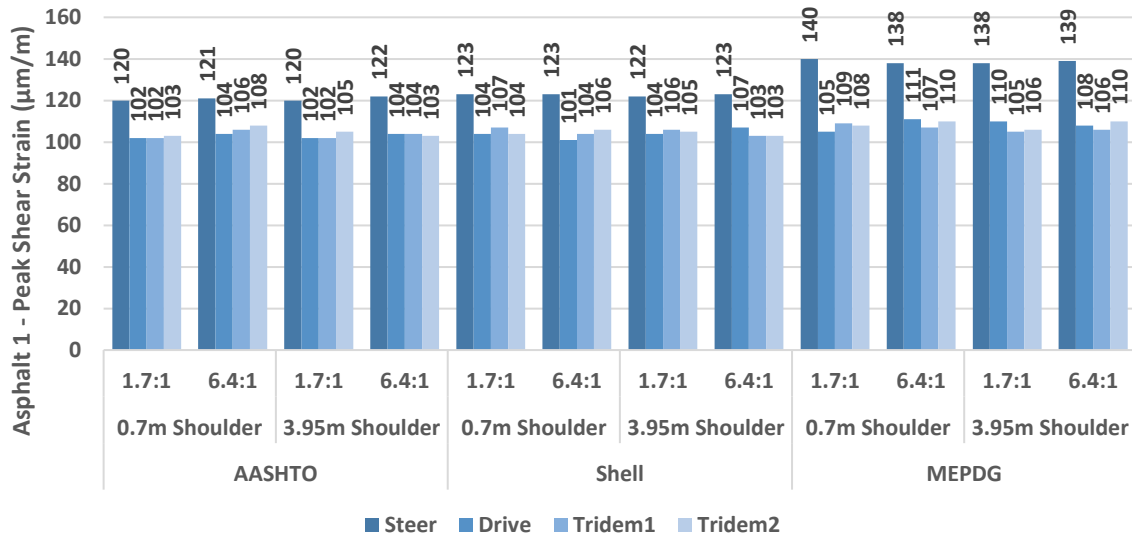


Figure 6. Peak Shear Strain in Asphalt Surface Layer (µm/m)

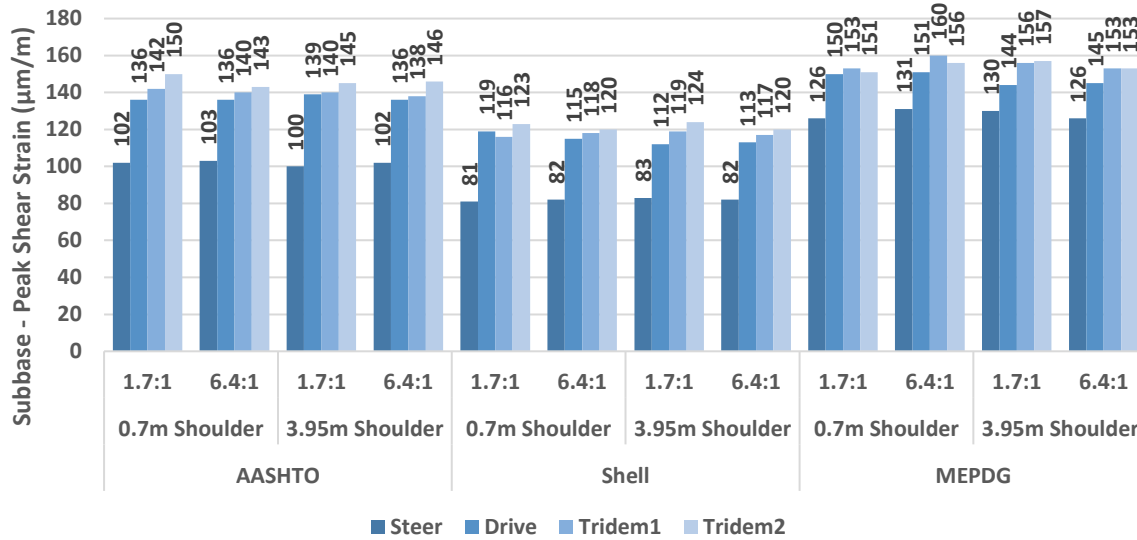


Figure 7. Peak Shear Strain in Granular Subbase (µm/m)

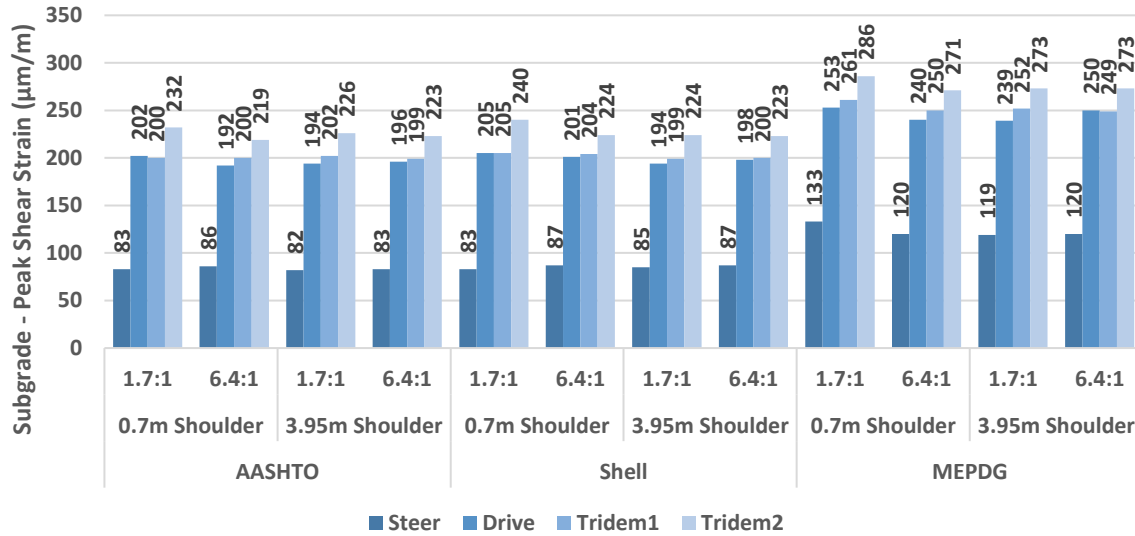


Figure 8. Peak Shear Strain in Silty Clay Subgrade ( $\mu\text{m/m}$ )

The following trends were observed in the asphalt, subbase and subgrade shear strain data:

- The shear strains in the asphalt surface layer reduced as the total thickness of the pavement structure increased. The area under the steer axle experienced the peak shear strain. For the other layers shown, areas under the tridem axle experienced the peak shear strain.
- The shear strains in the subbase layer did not follow the same trend as the surface layer. The subbase layer of Shell experiences lower shear strains than AASHTO 93 due to an additional 110 mm of Granular A in the base. This extra thickness shows a noticeable contribution in carrying the load.
- The shear strains in the subgrade layer decreased from MEPDG to Shell and AASHTO 93. The shear strains in AASHTO 93 and Shell are similar in the subgrade which indicates that even though the distribution of Granular is different, the performance is roughly equivalent in the subgrade layer.
- The peak shear strains were higher in the subgrade layer than the asphalt layer.
  - The large difference in modulus between the asphalt layers, subbase and subgrade layers causes a localized spike in shear strain between these layers.
- Geometry of the road does not have a major impact in the asphalt layers and subbase as shown in Figure 6 and Figure 7. As shoulder widths narrow, they cause an increase in the shear strains in the subgrade layer.
  - The shear strains are increased as the slopes are steeper for the narrow shoulder width (0.7 m). This is most evident in the MEPDG design.

Two examples of the contours generated by the 12 cases are presented in Figure 9 and Figure 10.



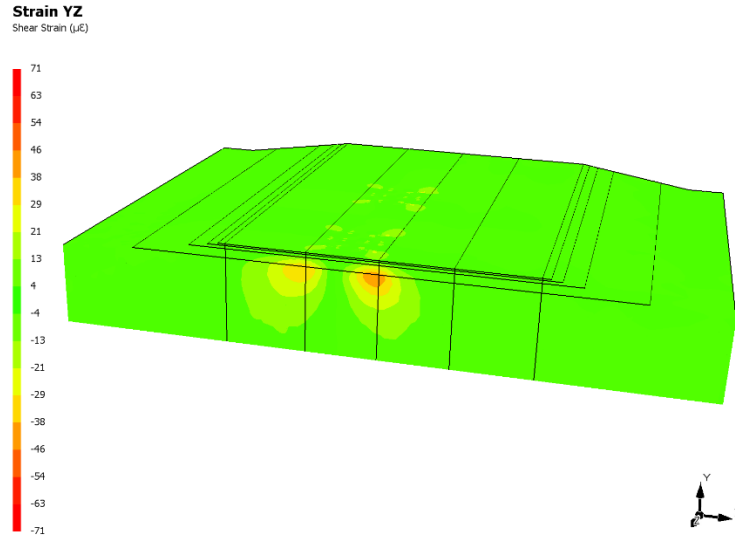


Figure 9. Shear Strain Contour for AASHTO 93 Thicknesses, 3.95 m Shoulder, 6.4:1 Side slope

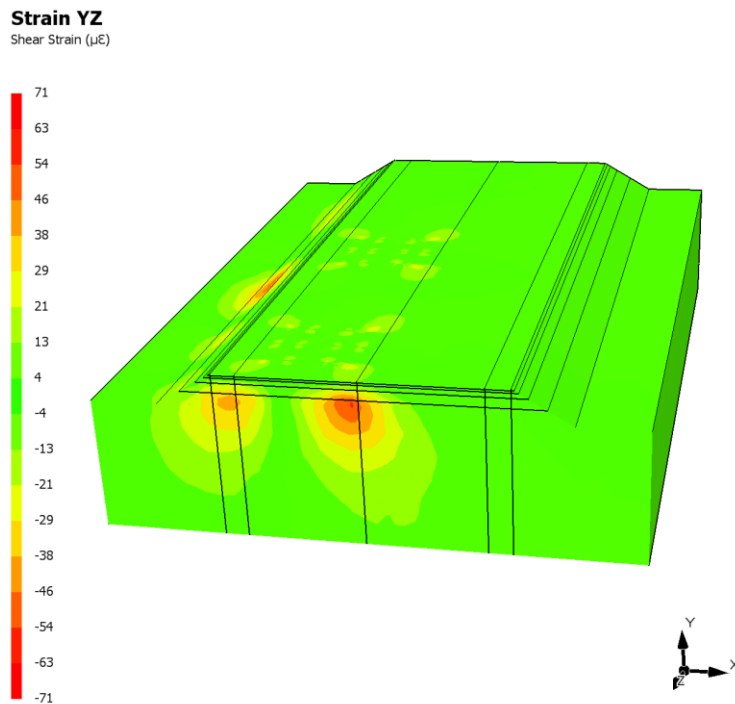


Figure 10. Shear Strain Contour for MEPDG Thicknesses, 0.7 m Shoulder, 1.7:1 Side slope

The three-dimensional contours shown in these figures highlight the shear strains in the pavement under traffic loading. The cross section shown located beneath the tridem load. The contour scale goes from green ( $0 \mu\text{m}/\text{m}$ ) to red ( $\pm 71 \mu\text{m}/\text{m}$ ).

- The case illustrated in Figure 9 has the greatest pavement structure thickness, the widest shoulders, and the gentlest slopes. As shown in Figure 9 the response in the pavement structure is minimal.

- In comparison, the case illustrated in Figure 10 has the least pavement structure thickness, narrow shoulders and steep slopes. Higher shear strains were observed under the load for the MEPDG case because of a thinner pavement structure, but the major difference is shown in the side slopes. In the case of MEPDG, the shear strains on the side slope are prominent, whereas in the AASHTO case there is minimal strain.

The response from the pavement structure identifies the effect that road geometry has on pavement performance and behaviour. The strains experienced in thinner pavement sections are greater and peak deflections are also greater.

The loading from the truck is observed to produce higher shear strains throughout the width of the loaded lane, in the shoulder and side slope for the thinner/narrower structure. Therefore, additional investigation of the shear strain in the subgrade side slope was investigated as illustrated in Figure 11. The impact of road geometry is most evident in the subgrade side slope below. The investigation of the side slope is important as high strains could lead to cracking along the edges of the pavement structure which eventually can propagate to the traffic lane and cause further issues.

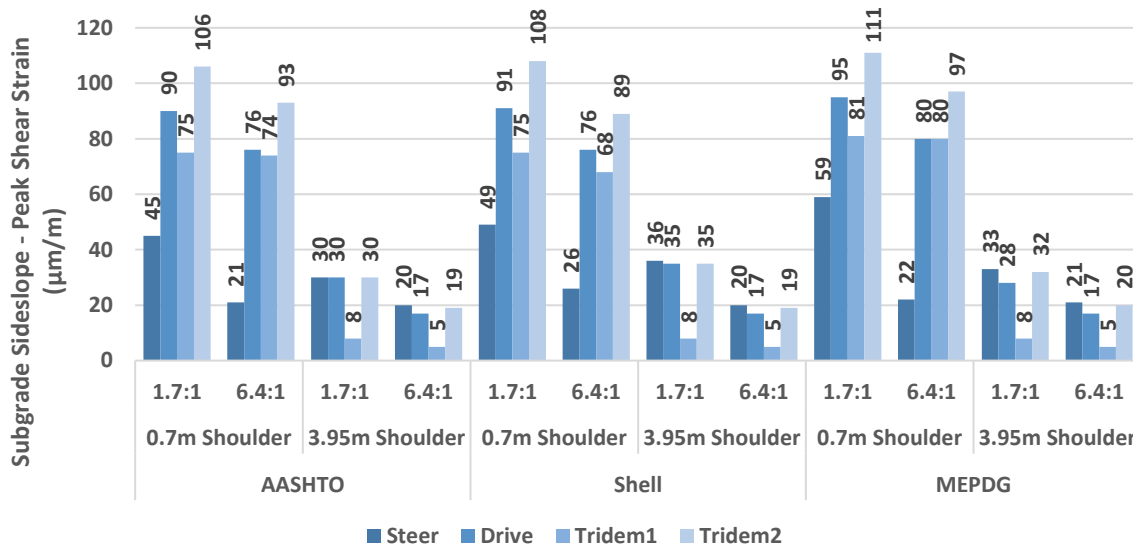


Figure 11. Peak Shear Strain in Subgrade Side Slope (µm/m)

As seen in Figure 11, the following trends were observed in the subgrade side slope:

- The shear strain in the subgrade side slope is dependent on the width of the shoulder and the side slope.
- Comparing the results within the same layer thickness cases, the difference in shear strain between the narrowest cases (narrow shoulders, steep side slope) and widest cases (wide shoulder and gradual side slope) increased by 520%, 530% and 550% for the AASHTO 93, Shell, and MEPDG thickness cases, respectively. PSIPave 3D™ can show the effect of these factors which other design methods do not account for.
- Therefore, geometry has a major effect on the subgrade side slope and these strains can cause failures in the side slope damaging the integrity of the overall pavement structure. Further issues such as erosion and entrance of moisture can take place which reduce the pavement performance. Figure 11 shows the importance of building wider shoulders to effectively reduce the shear strains in these cases.

## 6 CONCLUSIONS

This work highlighted that each pavement design method: AASHTO 93; Shell; and MEPDG could be used to develop a reconstruction pavement structure for North Service Road. The required inputs varied between methods and MEPDG required more data than the other two. Relatively similar pavement structures were developed by all three, with MEPDG allowing for a thinner layer of asphalt. However, none of these three methods could demonstrate the effect of road geometry or the behaviour of the materials within the pavement structure. PSIPave 3D™ provided information regarding the effects of road geometry and material behaviour. The distribution of stresses and strains throughout the pavement structure, in all orthogonal and shear directions was quantified by PSIPave 3D™. This information allows the user to re-evaluate their design and enhance it to create a pavement structure and road geometry that is more effectively handling the site conditions and loading. The following conclusions were observed:

1. The peak tensile strains observed in the MEPDG design are on average 24% more than AASHTO 93 and 22% more than Shell. The compressive strains observed in MEPDG are on average 23% more than AASHTO 93 and 21% more than Shell. Overall, the road geometry has minimal effects on the peak tensile strain and compressive strain.
2. The peak shear strains in the surface layer decreased as total pavement thickness increased. For the truck configuration utilized in the analysis, the steer axle induced the greatest peak shear strain near the surface. Geometry of the road did not have a major impact in the asphalt and subbase layers for peak shear strain. In the subgrade, the steeper side slopes had higher shear strains for the narrow shoulder width (0.7 m) most evident in MEPDG.
  - a. The large difference in modulus between the asphalt layers, subbase and subgrade layers causes a localized spike in shear strain between these layers.
  - b. The subbase layer of Shell experiences lower shear strains than AASHTO 93 due to a 110 mm increased in Granular A thickness on the layer above it. This extra thickness shows a noticeable contribution in carrying the load.
3. The greatest impact of geometry was shown in the subgrade side slope. Comparing the results within the same layer thickness cases, the difference in shear strain between the narrowest cases (narrow shoulder, steep side slope) and widest cases (wide shoulder and gradual side slope) increased by 520%, 530% and 550% for the AASHTO 93, Shell, and MEPDG thickness cases, respectively.
4. PSIPave 3D™ can determine the contribution of each axle to the peak normal strains, shear strains, and displacements which is something that conventional pavement methods using Equivalent Single Axle Load (ESAL) as the traffic input cannot show

Based on these findings, geometry had a small impact on the normal and shear strains under the load but showed a drastic effect in the subgrade side slope. High shear strains can cause failures in the side slope damaging the integrity of the overall pavement structure. These effects are not accounted for by conventional design methods. Therefore, wider shoulders could be considered to reduce the shear strains in the side slope.

## 7 FUTURE WORK

Future work would involve a variability study where the pavement structure is investigated at upper and lower limits of thickness to check for convergence.

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