Re-exploring the AASHTO 1993 Method for a Cost-Effective Pavement Design in Manitoba

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Abstract

Pavement structures costs constitute to the majority of the total costs of highway construction projects. Therefore, it is important to optimize each pavement structure to avoid an under-deign or overspending on any project. In the past, Manitoba was using the Benkelman Beam Rebound (BBR) deflection method for the rehabilitation design. A mixed approach, together with several environmental and structural adjustments, was used in pavement design for the new construction or reconstruction projects. The assumed or estimated values of subgrade and layer materials stiffness did not well represented the materials those are in place or use in Manitoba. These led to an overdesign for most rehabilitation and some new construction projects.

Due to several limitations of the AASHTOWare Pavement ME Design approach, that yet to be resolved, Manitoba has undertaken major changes to its existing design practices for cost-effective pavement structures. These include the use of more reliable/reasonable design traffic loading, layer materials and subgrade stiffness and drainage properties, pavement drainage condition, subgrade soils frost susceptibility, serviceability and reliability. As a result, significant cost savings are being realized. This paper presents an overview of Manitoba's new approach and outcome to share with other agencies, designers and students.

Introduction

Pavement structures costs constitute the majority of the total costs of all highway construction and rehabilitation projects. Manitoba Infrastructure (MI) has been struggling to maintain or improve the existing network health due to a limited budget, major capital investments to improve mobility through and within major urban centres, such as the new construction or reconstruction of interchanges, and the expansion of network size. A reduction in pavement structure thickness corresponds to a reduction in construction cost and contributes to better management of network health. However, such reduction of thickness may result in structurally inadequate pavement and reduction in service life. Therefore, it is important to optimize pavement structure thickness to achieve the desired performance or service life without overspending on any project.

Over the past several decades, MI was using the BBR deflection method for rehabilitation design of existing bituminous (asphalt concrete or AC) and asphalt surface treated (AST) pavements. MI was collecting the BBR data during the spring season of each year. These BBR data represent the weakest condition of pavements. Since the spring weak condition lasts for about two months in each year, these BBR data do not represent the annual average condition of pavement structure and subgrade. This led to overdesigns for most of the rehabilitation projects. The year-to-year variation of spring condition is also a major issue with the BBR data, in addition to its poor repeatability.

For the new construction of flexible, composite (AC over concrete) and rigid pavements, and rehabilitation of composite and rigid pavements, AASHTO 1993 design guide approach was in use to calculate the total structural number (SN). However, MI was calculating the design traffic loading, in terms of Equivalent Single Axle Load (ESAL), using the Modified Shell equation. This resulted in a very high design ESAL values as compared to the ESALs based on the AASHTO 1993 design guide. On the other hand, the assumed values of subgrade resilient moduli and pavement layer coefficients were relatively high. They did not represented well the materials that are in place or in use on Manitoba highways. In addition, several adjustment factors to SN were applied to account for the subgrade soil frost susceptibility, organic content and saturation, and highway x-section type. These adjustments resulted in overdesign in some cases.

Although MI is one of the leading agency in Canada in terms of evaluating the AASHTOWare Pavement ME Design [1] approach, MI slowed its implementation due to several major limitations that are yet to be resolved. The limitations include: low sensitivity to subgrade stiffness, no or low sensitivity to unbound granular material layers (thickness and stiffness) and high values of predicted longitudinal cracking. To make highway construction more cost-effective, MI started re-exploring the widely accepted/used AASHTO 1993 Guide [2] approach and undertaken major changes to its design practices. Changes include: 1) the use of project specific AASHTO 1993 ESAL value that varies based on truck class distribution and highway loading class instead of Shell ESAL; 2) the use of asphalt layer coefficient specific to local asphalt material; 3) the use of unbound base/subbase layers coefficients specific to local material, drainage and seasonal condition; 4) the use of appropriate subgrade moduli considering seasonal variation and drainage condition; 5) appropriate adjustment or management strategy for subgrade soils frost susceptibility; 6) discontinuation of BBR method; and 7) the use of more reasonable reliability and serviceability values. These resulted in significant reduction in pavement structures and cost savings.

Objective and Significance

The objective of this paper is to share MI's fresh approach with other agencies and designers, generate discussion and learn from each other. This paper, presentation and discussion is also expected to be an educational opportunity for new engineers, students and other interested individuals or agencies.

Pavement Design Considerations

Pavements are horizontal structures constructed on prepared subgrades, termed as the foundations of pavement structures, to carry roadway traffic loadings. A pavement structure should be sufficiently stiff or thick to distribute the imposed traffic load over a wide area of subgrade to limit the stress on the subgrade and avoid premature pavement distresses. In fact, pavements are generally layered structures consistent of several layer materials. Each underlying layer act as the foundation for the overlying layer(s). Each layer undergoes traffic and environment related stresses. Therefore, each layer should be sufficiently stiff or thick to avoid overstressing, and premature surface and layer distresses.

US FHWA Policy Guide [3] states that pavement structures shall be designed to accommodate the current and predicted traffic needs in a safe, durable and cost-effective manner. The main factors that a highway agency should pay particular attention when a designing pavement include traffic loads, materials, climate, drainage, construction practices, and desired performance over the design life. As pavements are built to facilitate traffic movement, accurate estimate of traffic loadings over the design life is extremely important. The design traffic loading should represent the current truck classification, weight and growth.

Since the load from traffic is ultimately transferred to the subgrade, a uniform, stiff, moisture and frost resistant foundation is the most important aspect of pavement structural design. A non-frost susceptible and free draining granular subbase layer should be considered for cold climate with frost penetration problem. The base layer should also be free draining or be resistant to moisture and stress related damage. Provision of adequate drainage and accounting for pavement structure drainage properties are important factors to ensure the desired pavement performance [3].

For rehabilitation design, it is essential that each project be properly engineered to ensure the best return for the money expended. This includes: 1) determining the condition of the existing pavement including proper identification of different types of distresses and their reasons; 2) environmental conditions; 3) layer material strength; and 4) layer material quality. The rehabilitation treatment should address the observed distress and its reason to prevent premature reoccurrence [3].

Pavement Design Methods

Many design approaches are use in North America and elsewhere. Some agencies follow the design approaches developed by National Transportation Associations such as Transportation Association of Canada (TAC) and American Association of State Highway and Transportation Officials (AASHTO) or Industry Associations such as Asphalt Institute (AI) and Portland Cement Association (PCA) with or without any local modification. Some agencies have developed their own agency specific design methods, e.g. CalME in California, MnPAVE in Minnesota. The Canadian Pavement Asset Design and Management Guide or PADMG [4] provided an excellent overview of different design approaches, inputs and requirements. However, it lacks sufficient details for a day-to-day design by local highway agencies. The AASHTO 1993 Design Guide [2], an empirical method, is still the most widely used pavement design approach. This is due to the experience and comfort that are developed by agencies over the last several decades. The AASHTOWare Pavement ME Design Program [1], developed based on the mechanistic-empirical approach and is still undergoing major changes, re-calibration and refinement, is not yet quite ready for full implementation by most highway agencies.

Manitoba's Design Inputs, Adjustments and Issues

Traffic Data

MI maintains a network level total traffic and truck traffic databases. Representative axle and vehicle weights data were collected in 2002 [5, 6] at different weigh scales. The Modified Shell equation was used to calculate the ESALs per axle and ESALs per truck, and then to develop the truck factors, which are the weighted average ESALs per truck for the mixed traffic on different classes of highway. These truck factors were in use in the past for pavement designs using both BBR deflection and AASHTO 1993 methods.

While the use of Modified Shell Equation to calculate ESAls per axle may be justified for use with the BBR method, its use with the AASHTO 1993 [2] approach does not appear to be a sound. The reason is that the structural requirement using the AASHTO 1993 design guide is based on the AASHTO 1993 ESAL factors. Figure 1 shows a comparison of ESAL factors between Modified Shell and AASHTO 1993 methods. As shown in the figure, the ESAL values from AASHTO table are exactly the same as ESAL values based on Shell equation for all weights on the single axle. However, AASHTO ESAL values for tandem and tridem (triple) axles are lower than the Shell ESAL values. Moreover, AASHTO considers steering axle as a single axle. The Shell steering axle ESAL values are significantly higher than the single axle ESAL values.

Table 1 shows a comparison of truck factors between two approaches. As per Table 1, Shell truck factors are twice the values that are obtained using the AASHTO ESAL tables. This issue, together with the abandonment of BBR method, triggered the development of new ESAL values for different trucks based on the AASHTO 1993 ESAL tables. This contributed to a cost-effective and sounder pavement designs.

Manitoba was using a 2% growth rate for forecasting the future traffic on all highways. Local experience has shown a higher growth rate on some highways, lower on many others and a negative growth on some highways. An 80:20 truck distribution between travel and passing lanes was used for four-lane divided highways without any local data analysis. These triggered the development of growth rate for different highways, assignment of more appropriate growth rate to each segment of the highway network and the development of lane distribution factors for divided highways.

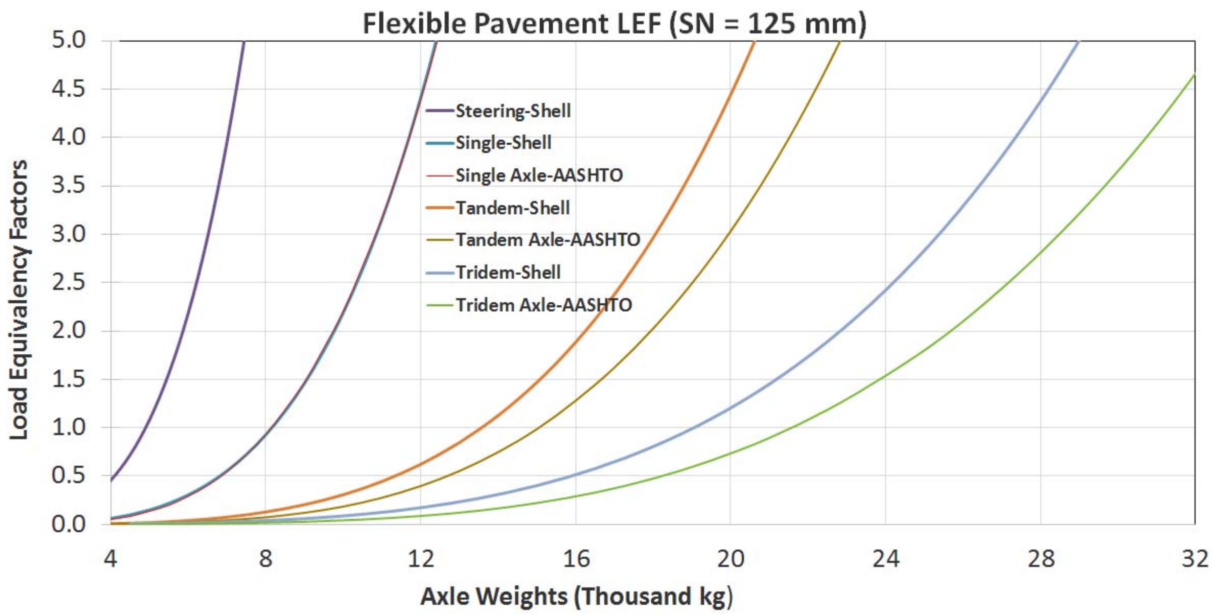

Figure 1. Comparison of ESALs for different axles between Modified Shell and AASHTO 1993 approaches.

Table 1. Comparison of truck factors between Modified Shell and AASHTO 1993 approaches.

Highway Classification	Shell ESALs Per Truck	AASHTO 1993 ESALs per Truck
PR- B1 Loading	2.00	1.06
PR- A1 Loading	2.30	1.20
PTH- A1 Loading	2.50	1.27
PR- RTAC Loading	2.55	1.34
PTH- RTAC Loading	3.05	1.41
NHS RTAC Loading	3.30	1.55

Subgrade Soil Classification and Resilient Modulus

Manitoba had been using the following formula to calculate the group index (GI) value for different subgrade soils. The GI value ranged from zero for fine sand to 20 for high plastic clay soils.

Group Index (GI) = (F1 - 35) [0.2 + 0.005 (LL - 40)] + [0.01 (F2 - 15) (PI - 10)] (1)

Where,

F1 = % passing 75 µm sieve, greater than or equal to 35% but to a maximum of 75% expressed as a whole number;

F2 = % passing 75 µm sieve, greater than or equal to 15% but to a maximum of 55%, expressed as a whole number;

LL = liquid limit, greater than 40 but to a maximum of 60; and

PI = plasticity index, greater than 10 but to a maximum of 30.

A resilient modulus (M_R) value was assigned to each soil class based on limited laboratory testing in an external laboratory. Table 2 shows the M_R of different soils that were used in the past. Based on further laboratory testing at the University of Manitoba [7] and FWD data, these M_R values were found to be high in most cases. The measured M_R values, at the University of Manitoba laboratory, varied from 50 MPa to 80 MPa at the optimum moisture contents for different soils. A higher difference between the measured and previously used M_R values was observed for stiffer soils (lower GI values). Some of the past subgrade M_R values were even higher than the measured M_R values for granular base [140 MPa, Ref. 8] and subbase materials that are in use in Manitoba. This issue was another major trigger for the changes in designs.

Table 2. Recommended resilient modulus (M_R) values for Manitoba subgrade soils [6].

Example of Soil Type	Group Index	M_R (MPa)	
		Southern Manitoba	**Northern Manitoba**
High Plastic Clay	20	30 – 35	35 – 40
High Plastic Clay	18	35 – 40	40 – 45
High/Low Plastic Clay	16	40 – 45	45 – 50
Low Plastic Clay	14	45 – 50	50 – 55
Low Plastic Clay	12	50 – 55	55 – 60
Silty Clay	10	55 – 60	60 – 65
Clayey Silt	8	65 – 70	70 – 75
Sandy Silt	6	80 – 85	85 – 90
Sandy Silt	4	90 – 95	95 – 100
Silty Sand	2	125 – 135	130 – 140
Fine Sand	0	150 – 200	150 – 210

Design Reliability

In Manitoba, the selected deign reliability was a function of x-section type (urban versus rural) and highway functional class. It varied from 80% to 90%. An 80% reliability for gravel roads and low volume surfaced road are considered to be high. A change was desired to reduce the construction costs for these low volume surfaced and unsurfaced roads.

Pavement Serviceability

Manitoba was using 4.5 as the initial pavement serviceability index (PSI) value for all highways regardless of quality of construction and initial pavement surface smoothness. The terminal PSI was 2.5 regardless of highway classes and traffic volume. A change was required to reflect the quality of construction and ride that are actually being achieved when selecting the initial PSI. A decision was also made to use lower terminal PSI values for low volume and secondary highways to save some money. This will improve the network health as savings from secondary or low volume highways can be invested to other highways.

Design Adjustments

In the past, Manitoba was adjusting the calculated structural number (SN) for frost susceptible subgrade soils, organic content (OC) in subgrade soils, x-section type and surface drainage. The calculated SNs based on traffic loading and subgrade stiffness were increased by 25% for frost susceptible soils (see author's discussion on frost susceptible soils and design consideration elsewhere). The increase in SN for organic content in soils varied from zero to 40% depending on the depth to organic layer, organic layer thickness, organic layer continuity and percentage of organics, as shown in Table 3. SNs were increased by 0%, 10% and 20% for rural (ditch depth >0.90 m), semi-urban (ditch depth >0.3 m but <0.9 m) and urban (ditch depth <0.30 m) cross sections, respectively. SN was further increased by 10% for ineffective (poor) surface drainage, high water table, or very wet subgrade. The historical basis for these adjustments is unknown. Therefore, a revisit was desired to make the design approach informed and technically more sound.

Table 3: Adjustment for Organic Content (OC) [6].

Zone	Depth Below Design Subgrade (mm)	Organic Content (%)	Note	Organic Condition	OC
Sub-cut	0 – 600	4-6	A	Discontinuous, randomised layers	0.1
		4-6	B	Continuous layers ≥100 mm thick	0.2
		7-10	A	Discontinuous, randomised layers	0.2
		7-10	B	Continuous layers ≥100 mm thick	0.4
		11 or more		At least some distinct deposits	Excavate
Below Sub-cut	600 - 1200	7-10	A	Discontinuous, randomised layers	0.1
		7-10	B	Continuous layers ≥200 mm thick	0.2
		11 or more	A	Discontinuous, randomised layers	0.3
		11 or more	B	Continuous layers ≥200 mm thick	0.4
		11 or more		Deposits ≥300 mm thick	excavate
	1200 - 1800	11 or more		Continuous layers ≥200 mm thick	0.2
		11 or more		Deposits ≥300 mm thick	0.4

Notes: (A) Not the preferred design option; (B) only as a last resort option.

Pavement Materials Stiffness and Layer Coefficients

MI was using layer coefficient values of 0.42 for asphalt concrete (Bituminous B), 0.14 for granular base (A base) and 0.12 for granular subbase (C base). These values are comparable to that used by other highway agencies in Canada and United States, and the design example in AASHTO 1993 [2] guide. However, laboratory testing at the University of Manitoba, to characterize Manitoba asphalt mixes [9], showed that the asphalt mix currently used in Manitoba, called Bituminous Mix B or Bit. B is a fine graded mix with a low elastic modulus. The average resilient modulus was ~2,500 MPa at 20 °C based on over 30 core samples that were collected between 2011 and 2017 from different project sites. This corresponds to a layer coefficient value of 0.40 for Manitoba Bit. B mix.

Like the bituminous mix, Manitoba unbound granular base (called A base) materials were also found to be fine graded with low modulus and poor drainage characteristics [8]. Laboratory testing at the University of Manitoba [8] on unbound granular base (limestone, gravel and granite) samples showed an average resilient modulus of 140 MPa at the optimum moisture content with poor drainage properties. This corresponds to layer coefficient value of 0.096. Further testing at the MI central laboratory showed average California Bearing Ratio (CBR) values of 35% for gravel A base samples and 45% for limestone A base samples. These correspond to layer coefficient values of 0.10 for gravel A base and 0.11 for limestone A base. CBR value for gravel A base containing ~5% shale particles, by weight, was 15%. This corresponds to a layer coefficient value of 0.069. For Manitoba granite subbase (C base), the CBR value was 21%. This corresponds to a layer coefficient value of 0.095. For the 50 mm minus crushed rock subbase, a layer coefficient value of 0.12 was used in the past. This rock material appears to be stiffer than A base that are in use in Manitoba. More testing on different materials is in progress at the MI's central laboratory.

Existing Pavement Structural Capacity for Rehabilitation Design

For the rehabilitation design, the BBR deflection was in use until 2016 to estimate the strength and load carrying capacity of the existing flexible pavements. To account for the variation from year to year and within a year, the latest 10 years of data were used to calculate the representative BBR. However, BBR data were collected during the spring thawing season that reflects the pavements weakest condition within a year. This weakest spring condition lasts for about two months in each year and the data did not reflect the annual average condition. This led to overdesign in most cases of rehabilitation projects. In addition, poor repeatability of BBR data is as issue.

MI purchased a falling weight deflectometer (FWD) in 2008 and discontinued the BBR data collection. Between 2009 and 2016, Manitoba collected FWD deflection data from 87% of the paved network. The second round of network FWD testing was started in 2017 and ~49% of the paved network has been completed to date. All FWD data are being collected during the summer/fall months (June to October). Project level FWD data are also collected for each upcoming construction projects.

Changes to Manitoba's Design Inputs and Process

Traffic Data and Design Loading

Manitoba currently maintains several weigh in motion (WIM) stations. In 2010, Manitoba developed axle load spectra [9] and updated them in 2016 [10] using data from the WIMs. The WIM data are used to develop the new ESAL factors for each axle types and then the ESALs per truck for each truck classes. These two traffic studies also developed truck class distribution for each sections of the entire paved highway network. The later study also included the truck growth rates for each road segment of the entire paved network. The calculation of truck factor is now section (project) specific. Table 4 presents an example of truck factor calculation for an RTAC route based on truck class distribution and ESAL per truck for individual truck classes. The calculated accumulated design life ESALs using the new ESAL factors closely matches with that calculated using Pavement ME Design software for SN 5 (125 mm).

The selection of a design ESAL values for pavement design using the AASHTO guide is very cumbersome as it depends on design SN and terminal serviceability, in addition to axle types and axle loads. Considering the extra iterations and time, that are required to match the assumed SN and calculated SN for the selection of truck factor, Manitoba decided to use ESAL factors based on a lower Pt and SN for secondary highways, and a higher Pt and SN for primary highways as shown in Table 4.

The annual growth rate for the truck traffic varies from a negative value to 4%, with a few exceptions. For design purposes, zero percent growth rate is being used when the data shows a negative growth i.e., a decrease in truck volume. The directional spilt of traffic varies on some locations but it is generally 50:50. The percentage of trucks on the travel lane for four-lane divided highways varied from 77% to 97% with an average of 90%. For design purposes, 80% is now being used for perimeter highways that encircles City of Winnipeg and 90% is being used for other four-lane divided highways.

Table 4. Calculation (example) of ESAL per truck for mixed traffic.

Serviceability		2.00	2.00	2.25	2.25	2.50	2.50
Apply to Highways		B1-Gravel	B1-AST	B1- Asphalt	A1	RTAC	RTAC-NHS
Vehicle Class	Class Dist., %	SN 1	SN 2	SN 3	SN 4	SN 5	SN 6
4	0.80	1.103	1.123	1.177	1.159	1.142	1.119
5	3.61	0.492	0.500	0.508	0.483	0.465	0.464
6	3.62	0.634	0.644	0.666	0.649	0.636	0.630
7	0.12	1.176	1.188	1.202	1.174	1.153	1.150
8	2.30	0.605	0.621	0.646	0.611	0.587	0.579
9	46.94	1.036	1.063	1.124	1.098	1.078	1.053
10	20.37	1.214	1.227	1.271	1.235	1.226	1.210
11	0.27	0.594	0.623	0.678	0.645	0.625	0.603
12	0.82	0.856	0.899	0.997	0.957	0.927	0.885
13	21.15	2.252	2.281	2.367	2.315	2.292	2.260
Weighted Avg. ESALs/Vehicle						***1.314***	

Subgrade Soil Classification and Resilient Modulus

Manitoba conducts project level coring to determine type, thickness and condition of each layer of pavements for all reconstruction and rehabilitation projects. Soil survey, sampling and testing are done to determine the soil type/classification and soil contents for all projects including the new construction. Subgrade soils are classified according to AASHTO and Unified Soil Classification systems.

Project level FWD deflection data are also collected during the summer months from capital projects that are upcoming in the following year. The project level FWD data are being used to determine the resilient moduli of subgrade soils. The soil properties are primarily used to check the reasonableness of the resilient modulus obtained from the backcalculation of FWD data and to determine the frost susceptibility of subgrade soils. For the new construction, backcalculated modulus from adjacent highway section or lane is being used provided that soil contents and classification are similar. If no FWD data is available, estimated value of M_R is used, Designs are considered preliminary when estimated M_R values are used. Preliminary designs are good for budgeting purposes, but not good for construction.

MI collected FWD data from several research sites in Manitoba between 2009 and 2012 in different months. ELMOD backcalculation software was then used to determine the pavement layers and subgrade moduli from these FWD deflection basin data. The backcalculated subgrade moduli for different months were used to establish factors for the seasonal variation of subgrade soils resilient modulus (further review is in progress). However, the backcalculated modulus from ELMOD software may not be appropriate for use in pavement design using the AASHTO 93 guide as significant difference in M_R between AASHTO 93 [2] and ELMOD was recorded. Therefore, to determine the summer/fall M_R of subgrade soils, the following AASHTO 1993 Guide [2] equation has been adopted.

$$M_R = 0.24P/(d_r*r) \quad (2)$$

Where,

P = applied load in psi,

d_r = deflection in inches at radial distance r (corrected to 20 °C), and

r = radial distance at which deflection is measured

The standard 40 KN load (566 KPa stress) is applied to collect the FWD deflection basin data. The deflection measured at a radial distance of 1,200 mm from the centre of the plate is being used to backcalculate the subgrade $M_{R.}$ As per the guidance provided in FHWA-RD-97-077 [11], a correction factor of 0.35 is used to convert the backcalculated M_R to the equivalent laboratory measured M_R.

Table 5 shows a sample calculation of effective M_R based on summer/fall FWD data and seasonal factors for a sandy clay subgrade. The seasonal factors vary depending on drainage condition and moisture exposure. For example, if subgrade is exposed to moisture approaching saturation in October, the seasonal factor will be reduced to 0.5 for October, unless FWD testing is done in October. If the subgrade is saturated year round, excluding the winter freezing period, the factors for April to November will be 1.0 for southern Manitoba (Zone 1).

Table 5. Example of subgrade effective resilient modulus calculation (PTH 21 @ US Border).

Zone (Area) Code =	1	Seasonal M_R, psi	Seasonal Rel. Damage, U_f	Selected Seasonal Factors		
Summer M_R, psi =	7,544			MB Climate Zones		
Month	Factors			Zone = 1	Zone = 2	Zone = 3
January	6	45,263	0.00186	6.0	6.0	6.0
February	6	45,263	0.00186	6.0	6.0	6.0
March	3	22,632	0.00931	3.0	4.0	5.0
April	0.5	3,772	0.59463	0.5	0.5	0.5
May	0.5	3,772	0.59463	0.5	0.5	0.5
June	0.8	6,035	0.19984	0.8	0.8	0.8
July	1	7,544	0.11909	1.0	1.0	1.0
August	1	7,544	0.11909	1.0	1.0	1.0
September	1	7,544	0.11909	1.0	1.0	1.0
October	1	7,544	0.11909	1.0	1.0	1.0
November	1	7,544	0.11909	1.0	2.0	3.0
December	2	15,088	0.02385	2.0	3.0	4.0
Sum of Relative Damage			2.02143			
Average Relative Damage			0.16845			
Effective M_R, psi			6,496			
Effective M_R, MPa			44.8			

Design Reliability

The design reliability reflects confidence for pavement structure to remain at the desired serviceability level up to or exceeding the design life (desired initial pavement performance period). If a pavement structure fails to meet its design life, early maintenance or rehabilitation treatment will be required. This could a major issue for primary highways (high traffic), but not a very significant issue for secondary (low traffic) highways. Also fixing rural highways is easier than fixing the urban highways. Fixing thin surfaced or unsurfaced pavements is easier and less costly than thick/hard surfaces. Therefore, a lower reliability i.e., a higher risk may be taken for secondary highways, rural areas and thin or unsurfaced pavements.

This will save some money on these highways. Table 6 shows the selected design reliabilities for different highway classes, surface type and locations.

Table 6. Guideline for the selection of design reliabilities.

Highway Classification	Surface	Rural	Urban
Provincial Trunk Highway (PTH): Expressway	All	90	90
Provincial Trunk Highway (PTH): Non-expressway	All	85	90
Provincial Road (PR)	Bit./Concrete	80	85
Provincial Road (PR)	AST	70	80
Provincial Road (PR)	Gravel	50	60
Provincial Access Road (PA)	Bit.	70	80
Provincial Access Road (PA)	AST	60	70
Provincial Access Road (PA)	Gravel	50	60

Pavement Serviceability

The initial serviceability of a pavement is a function of construction quality. For example, an asphalt surface treated (AST) pavement or thin bituminous pavement cannot be constructed as smooth as a thick pavement with multiple asphalt lifts. Based on the post-construction relative smoothness data on Manitoba highway construction projects, the new initial serviceability guideline has been developed as shown in Table 7.

Table 7. Guideline for the selection of initial pavement serviceability index (P_i).

Surface Layer	**P_i** (International roughness index)
AST or Traffic Gravel	4.0 (Not available)
1 lift Bit.	4.1 (1.1 m/km)
2 lifts Bit.	4.2 (1.0 m/km)
3 lifts Bit.	4.3 (0.9 m/km)
4 lifts Bit.	4.4 (0.8 m/km)
>4 lifts Bit.	4.5 (0.7 m/km)

Table 8 shows the new guideline for pavement terminal serviceability index (P_t). This guideline is developed considering functional classes of highways and traffic volume. A higher P_t is desired for expressways and primary highways than low volume collector or local highways/roads.

Table 8. Guideline for the selection of terminal serviceability index (P_t).

Highway Classification	AADT	Pt (IRI)
Expressway	N/A	2.5 (2.5 m/km)
Primary Arterial	N/A	2.5 (2.5 m/km)
Secondary Arterial	N/A	2.4 (2.6 m/km)
Collector/Local	>2,000	2.3 (2.7 m/km)
Collector/Local	750-2,000	2.2 (2.8 m/km)
Collector/Local	250-750	2.1 (2.9 m/km)
Collector/Local	<250	2.0 (3.0 m/km)

Pavement Layer Materials Stiffness, Layer Coefficients and Drainage

As mentioned earlier, Manitoba's bituminous mix is fine graded with a low modulus value. Therefore, a layer coefficient of 0.40 has been selected for Manitoba Bit. B mix based on laboratory testing for resilient modulus of asphalt cores taken from highways projects. Technically, an effective resilient modulus and effective layer coefficient should be used considering variation of stiffness from month to month or season to season. The temperature sensitivity of the asphalt mixes make it difficult to establish representative value for any month or even for a given day. The accuracy of such effective modulus would be poor. Therefore, the layer coefficient at standard temperature (20 ^{o}C) has been considered the best available option. This value is conservative enough for Manitoba given that the annual average mean temperature is approximately 3 ^{o}C in Winnipeg, asphalt pavement temperatures are 0 to 10 ^{o}C higher than the air temperatures, and effective pavement temperatures are lower than the surface temperatures.

For the unbound granular base and subbase materials, AASHTO 93 guide [2] has provided equations and charts to estimate the layer coefficients from resilient modulus, CBR, R-value or Texas Triaxial value. AASHTO 93 also has provided guideline for adjusting unbound material layer coefficient using drainage coefficients (m-value). The m-value depends on drainage quality and percentage of time the layer in question is exposed to moisture approaching saturation moisture. For example, for a poor drainage quality and exposure to moisture approaching saturation for 5% of times, the m-value is 0.80. While the drainage quality of a layer material can be measured in the laboratory, estimation of percentage of time a layer is exposed to moisture approaching saturation is difficult. This may result in a very low effective layer coefficient. For example, effective layer coefficient of Manitoba A base will be 0.096*0.8 = 0.077. This will lead to a very thick pavement structure and it will be difficult to justify. Another major issue associated with the layer coefficient is that it does not account for the seasonal variation of layer stiffness and weakness. For an economic use of unbound granular materials, alternative approaches were explored including the development of a new granular base specification and effective value of layer coefficients.

Due to the poor drainage quality and low stiffness issues of Manitoba A base material, a new specification has been developed for a stiffer and better draining granular base material, designated as GBC-Type I. The laboratory constituted material showed a resilient modulus value in between 219 MPa and 288 MPa [12]. Testing at MI's Central laboratory on similar material collected from the field showed a CBR value of 100%. For a less stiff base with similar drainage quality, which was used under concrete pavement as a trial, the CBR value was 75% and the M_R value was 194 MPa. Considering all these test results, a resilient modulus value of 212 MPa, which corresponds to layer coefficient value of 0.14, was selected for this new material (GBC Type I) as an interim basis. Further testing and field trial are in progress to finalize the specification, stiffness and layer coefficient value for this new base material.

The FHWA repot FHWA-RD-97-077 [11] provided equations (Equations 3 and 4) to calculate the equivalent annual resilient modulus of unbound base and subbase layers. The calculated annual resilient modulus is to be used to determine the minimum thickness of bituminous layer using the AASHTO 93 design guide [2] to limit the tensile strain to an acceptable limit. Since the base and subbase layers are intermediate foundations for a surface layer, these equations account for the relative damage based on seasonal variation of base and subbase layers moduli.

$$U_f = 1.885*10^3*(M_R)^{-0.721} \tag{3}$$

$$M_R\ (Base) = [\Sigma\ (U_f)_i*(M_R)_i]/\Sigma(U_f)_i \tag{4}$$

Where,

U_f = Damage factor for a given M_R,

$(U_f)_i$ = Damage factor for season i,

$(M_R)_i$ = Resilient modulus in season i (psi), and

M_R (Base) = Equivalent annual resilient modulus of base

The calculated equivalent annual resilient moduli for unbound granular base and subbase materials can be used to estimate the structural layer coefficients if a large modulus ratio between adjacent layers does not occur. A high ratio can result in a high tensile stresses at the bottom of the base or subbase layers that tends to loosen the materials, which will then exhibit lower resilient moduli. The equivalent annual layer coefficients of granular base and subbase can be calculated using Equation 5 [11].

$$a_2 = 0.249*(\log_{10}M_R) - 0.977 \quad (5)$$

Where,

a_2 = Base or subbase layer coefficient

M_R = Resilient modulus of base or subbase

As mentioned earlier, MI has developed seasonal factors for subgrade soil resilient modulus using FWD data collected in different seasons from various research sites. The same dataset was used to develop the seasonal factors for the variation of base layer modulus in different times of the year. These factors are now used to estimate the equivalent annual modulus of base and subbase layers. Table 9 shows an example of equivalent M_R and layer coefficient calculation for granular A base. The summer modulus represents the modulus determined in the laboratory or estimated from CBR values.

The seasonal factors for base/subbase M_R varies based on: a) x-section type (urban, semi-urban or rural) that affects effective drainage; b) drainage quality of the material; and c) depth of layer (base versus subbase). The new drainable stable granular base stiffness is less susceptible to changes in moisture exposure and freezing than the A base. As the base layer is close to the surface, it is more exposed to moisture and freeze/thaw weakening than the subbase layer. As shown in Table 9, the equivalent annual layer coefficient is 0.115 for A base (as opposed to 0.077) and use of no drainage coefficients are required.

Table 10 shows a comparison of layer coefficients for different unbound materials. Further testing is underway to confirm the layer moduli and seasonal factors.

Existing Pavement Structural Capacity for Rehabilitation Design

AASHTO 1993 Guide [2] has provided three approaches to determine the load carrying capacity or structural number of the existing pavement, namely the remaining life; visual survey and estimate or measure layer coefficients; and non-destructive deflection test. In the remaining life approach, the existing pavement load carrying capacity is estimated based on the traffic loading to failures, traffic loading that the pavement already experienced, original pavement structural capacity and structural capacity after to date traffic exposure. Obtaining or estimating these information is a difficult task, if not impractical, and therefore this approach was considered unsuitable for Manitoba. In the second approach, the existing pavement SN is calculated based on each layer thickness and its layer coefficient. AASHTO 93 has provided guideline for subjective estimation of layer coefficient of in-situ asphalt layer based on the extent and severity of cracks. Laboratory testing for appropriate layer coefficient is resource intensive. The subjective estimation of layer coefficient for each layer is an option, but it may not be very accurate or dependable.

Table 9. Example of equivalent M_R and layer coefficient calculation for granular A base.

Cross-Section Type/Code		**Rural and Semi-urban = 1**	**Urban = 2**	Summer M_R (MPa)	**140**
X-Section Code	**1**			**Granular A base**	
Summer M_R, PSi	20,305			Selected Seasonal Factors	
Month	Factors	M_R, Psi	Damage Factors	Rural/Semi-urban	Urban
January	4	81,221	0.54380	4.00	3.00
February	4	81,221	0.54380	4.00	3.00
March	2	40,611	0.89636	2.00	1.50
April	0.6	12,183	2.13539	0.60	0.50
May	0.7	14,214	1.91077	0.70	0.70
June	0.8	16,244	1.73539	0.80	0.80
July	1	20,305	1.47749	1.00	1.00
August	1	20,305	1.47749	1.00	1.00
September	1	20,305	1.47749	1.00	1.00
October	0.8	16,244	1.73539	0.80	0.70
November	0.9	18,275	1.59410	0.90	0.80
December	3	60,916	0.66914	3.00	2.00
Sum Product			394189.52		
Sum U_f			16.20		
Equivalent M_R, Psi			24,338	**Eqv. Layer Coeff.**	
Equivalent M_R, MPa			**167.8**	**0.115**	

Table 10. Layer coefficients for different unbound granular materials.

Layer Material	**Summer M_R, MPa**	**New Layer Coefficient (past value)**	**Equivalent Annual Layer Coefficient**		
			Rural	**Semi-urban**	**Urban**
Pulverized Asphalt	N/A	0.14 (0.14)	0.140	0.140	0.140
GBC- Type I	**212**	0.14 (NA)	0.158	0.158	0.138
A Base	**140**	0.096 (0.14)	0.115	0.115	0.102
A Base with Shale	**105**	0.069 (0.14)	0.089	0.089	0.076
C base (subbase)	**90/105**	0.095 (0.12)	0.095	0.084	0.084

NA = Not applicable (new drainable- stable unbound granular base material)

In the third approach, the existing pavement structural capacity is estimated using the backcalculated effective modulus and thickness of entire pavement structure. The pavement effective modulus is estimated using backcalculated subgrade modulus from FWD deflection data, FWD central deflection at 20 ^{O}C, FWD plate radius, pressure on FWD plate and total thickness of pavement structure. This approach is considered to be more accurate than other approaches as the measured data, using a well accepted technology that accounts for the condition of the existing pavement, are used. Therefore, this third approach has been adopted to calculate the effective SN (SN_{eff}) of existing flexible pavements. The visual survey and layer coefficient method is being used as a crosscheck or for preliminary design when the FWD and coring data are unavailable. This new approach, that replaced the BBR method, is providing a thinner overlay than the past and significant cost savings.

Design Adjustments

Table 11 presents the comparison of design adjustments between the past and new practices. The management and adjustment for subgrade soils frost susceptibility issue has been presented in a separate paper. The new approach appears to be technically sounder with less assumption and the use of measured data for local materials and field conditions.

Table 11. Comparison of design adjustment between the past and new practice.

Factor	Past Practice	New Practice
Organics	0-40% Up	None: Accounted for by the effective M_R
Pavement drainage (cross section type)	0-20% Up	None: Use appropriate equivalent annual layer coefficients
Surface drainage and high water table	0-10% Up	None: Accounted for by the effective M_R and appropriate equivalent annual layer coefficients
Frost Susceptibility	25% Up	Manage frost problem or minimal increase*

**Details is discussed elsewhere*

Impact of Changes to Pavement Design Process: Design Examples

Design for New Construction and Reconstruction

There is no change in basic approach to determine the total design structural number (SN_{dgn}) from that was in use in the past, except the changes in the process of determining the input values and elimination of adjustments to SN_{dgn} as discussed earlier in this paper. The SN_{dgn} is calculated using the Manitoba's standard design template. The inputs are the new set values of design life ESALs, initial serviceability, terminal serviceability, design reliability, overall standard deviation and roadbed (subgrade) M_R.

Thickness of Asphalt Layer: In the past, the selection of minimum thickness of bituminous layer was based on annual Shell ESALs (<40,000 to >200,000) and highway loading class (B1, A1 and RTAC). It varied from 85 mm bituminous or asphalt surface treated (AST) to 125 mm bituminous. In 2016, the minimum requirement was revised and set based on the design life (20 years) ESALs. The requirement was increased to 150 mm for high traffic loading (10 to 29.9M Shell ESALs) and 200 mm for very high traffic loading (≥30M Shell ESALs). The requirements were AST for very low traffic (<0.3M Shell ESALs), 85-100 mm bituminous for low traffic loading (0.3-2.9M Shell ESALs) and 125-150 mm bituminous for moderate traffic loading (3-9.9M Shell ESALs). However, flexible pavements are layered structures and each layer thickness should be determined according to layer principle [2]. The base layer acts as an intermediate foundation for the load/stress in surface layer. The minimum bituminous thickness should be determined considering base layer as the subgrade (foundation).

On the other hand, the in-situ M_R of unbound granular base layer depends on the M_R of the subbase layer, which is placed directly below the base layer. A large ratio of moduli values between adjacent layers can produce a high tensile stress at the bottom of the base layer and reduction of base layer modulus due to de-compaction of base layer. The same problem applies to subbase layer when the subgrade is coarsegrained or non-cohesive material. Upper limits of moduli for unbound base and subbase layers are therefore used to avoid high tensile stresses to occur [11], that provides sufficient thickness of each layers. When determining the thickness of bituminous layer, base layer modulus should not exceed 275 MPa [2].

MI now uses the base layer equivalent annual modulus (not to exceed 275 MPa) to determine the structural number (SN_1) for the asphalt surface layer. This will reduce or limit the tensile strains in the asphalt concrete to an acceptable level. The required minimum asphalt thickness (D_1) is then determined by dividing SN_1 by the layer coefficient of base layer (a_1). The recommended asphalt layer thickness (D_a) is the required minimum plus additional for levelling (usually 12.5 mm), rounded to suit the lift thickness requirement. The lift thickness is usually 45 to 55 mm (but can be as low as 35 mm and as high as 60 mm for lower lifts). The actual asphalt layer thickness (D_a) is then used to calculate the actual structural number (SN_a) of the asphalt layer as $SN_a = (D_a - 12.5)*0.40$.

Thickness of Base Layer: In the past, the minimum thickness of unbound granular (base plus subbase) varied from 100 mm to 200 mm depending on subgrade soil classification and highway loading class. As subbase work as an intermediate foundation for the loading on base and surface layers, MI now determines the structural number (SN_2) required to support the base and surface layers using the equivalent annual modulus of subbase layer. The required structural number for the base layer (SN_b) is determined by subtracting SN_a from SN_2. The required thickness of the base layer (D_2) is then determined by dividing the SN_b by the equivalent annual layer coefficient (a_2) of the base layer. The minimum thickness of base layer is equal to the thickness of asphalt layer to avoid overstressing the base layer. The maximum thickness of base layer is three times the thickness of asphalt layer to avoid a large modulus ratio. The actual base layer thickness (D_b) that will be placed is then used to calculate the actual structural number (SN_{ba}) of the base layer as $SN_{ba} = D_b*a_2$.

Thickness of Subbase Layer: In the past, there was no minimum thickness specified for unbound granular subbase. The required structural number for the subbase layer (SN_{sb}) is now determined by subtracting the total actual structural number for asphalt and base layers ($SN_a + SN_{ba}$) from SN_{dgn}. The thickness of the subbase layer is then determined by dividing SN_{sb} by the equivalent annual layer coefficient (a_3) of the subbase layer. The minimum thickness of subbase layer is equal to the thickness of base layer to avoid overstressing the subbase layer. The maximum thickness of subbase layer is three times the thickness of base layer to avoid a large modulus ratio between base and subbase. The thickness is rounded to suitable lift thickness.

Design Examples: Table 12 presents two examples of the impact of changes from the past practice. Using the new practice, significant savings are achieved in terms of reduced pavement and embankment thickness for design example 1. Additional savings are achieved using the new granular base (GBC Type I).

For Design Example 2, there is no major savings in New Practice-1 in terms of design SN and total thickness because no adjustment for SN was required based on the past practice. However, significant thickness reduction has been achieved using the new granular base material. The required bituminous thickness increased from 150 mm to 165-190 mm as the minimum thickness of surface layer was estimated using modulus of base layer. Furthermore, the asphalt binder requirement for the surface lift changed from expensive PG 64-37 to relatively inexpensive PG 58-37. These resulted in significant cost savings.

Table 12. Design examples for new construction.

Attributes	Past practice	New practice-1	New practice- 2	Comments
Example 1: PTH 15 (Region 1), 250 truck/day, urban cross-section, Sandy clay soils with 9% organics				
TEF	2.5	1.022	1.022	60% reduction
20-year ESALs	2.7 million	1.1 million	1.1 million	60% reduction
Subgrade M_R	50 MPa	29.4 MPa	29.4 MPa	41% less
Serviceability	4.5/2.5	4.3/2.4	4.3/2.4	Initial/terminal
Reliability	85%	85%	85%	
Design SN	102 mm	109.2 mm	109.2 mm	7% higher
Adjustments	20+20 = 40%	Nil	Nil	For x-section and organics
Adjusted SN	144 mm	109.2 mm	109.2	24% savings
Surface (Bit. B)	150 mm	150 mm	150 mm	
A Base	200 mm	200 mm	*200 mm	*New granular base
C base	550 mm	400 mm	300 mm	150-250 mm less C base
Total Thickness	900 mm	750 mm	650 mm	17-28% less thickness
Example 2: PTH 2 (Region 2), 800 truck/day, rural cross-section, high plastic clay soils				
TEF	3.05	1.375	1.375	55% reduction
20-year ESALs	11.5 million	5.2 million	5.2 million	55% reduction
Subgrade M_R	30 MPa	**25 MPa	**25 MPa	**Based on CBR
Serviceability	4.5/2.5	4.3/2.5	4.3/2.5	Initial/terminal
Reliability	85%	85%	85%	
Design SN	142.6 mm	139.9 mm	139.9 mm	2.7% less
Adjustments	Nil	Nil	Nil	
Adjusted SN	142.6 mm	139.9 mm	139.9	2.7% savings
Surface (Bit. B)	150 mm	190 mm	165 mm	
A Base	650 mm	600 mm	*500 mm	*New granular base
Total Thickness	800 mm	790 mm	665 mm	1-17% less thickness
Asphalt Binder	PG 64-37/ PG 58-34	PG 58-37/ PG 58-34	PG 58-37/ PG 58-34	Surface lift/other lifts

Design for Pavement Rehabilitation

The determination of the existing pavement load carrying capacity in terms of effective structural number (SN_{eff}) is discussed earlier. The calculation of the design structural (SN_{dgn}) for the design traffic loading is the same as the new construction. The structural number for the required overlay (SN_{ol}) is the difference between the SN_{dgn} and SN_{eff}.

Straight Overlay: The thickness of asphalt overlay without any treatment of the existing asphalt layer is determined by dividing the SN_{ol} by layer coefficient (a_1) of asphalt layer. An additional 10-15 mm asphalt (Bit. B) is added as leveling requirement to the calculated overlay thickness, depending on the surface roughness and rut depth. The overlay thickness may be further increased slightly to meet the lift thickness requirement.

Mill and Overlay: The mill depth is determined based on asphalt layer condition, surface roughness and rut depth. The milled thickness is considered as a partial loss of the structural capacity of the existing pavement. This loss is compensated by adding SN loss (SN_{loss}) due to the milling to the design overlay SN (SN_{ol}). A good estimate of the existing asphalt layer elastic modulus is possible through backcalculation of FWD deflection basin or laboratory testing. This process is however time consuming and backcalculation lacks consistency. Moreover, AASHTO 93 guide was not developed to include such layer by layer modulus from backcalculation and it is not a recommended option. However, more knowledge has been gained over time about backcalculation tools and process. MI will consider such option in the near future to incorporate in the design using the AASHTO 1993, if Pavement ME Design program does not work reasonably. In the interim, reasonable estimate of layer coefficient for the existing asphalt is used based on visual evaluation of cores, aging of pavement layer and moisture related damage, and pavement surface condition data as shown in Table 13.

Table 13. Estimation of layer coefficient (a_{1ex}) of existing asphalt.

Surface/ Layer Conditions	**Visual Observation of Cores**	**Layer Coefficient (a_{1ex})**	
		Layer Removed	*Layer Remains in Place
Very Good	Few or no surface cracks. Asphalt matrix is well bonded and no sign of aging or moisture related damage.	Overlay, if needed	0.35
Good	Intermittent surface cracks. Asphalt matrix is well bonded but slightly aged. Slight moisture related damage.	0.35	0.3
Fair	Frequent surface cracks. Asphalt matrix is well bonded but moderately aged. Moderate moisture related damage.	0.30	0.25
Poor	Extensive surface cracks. Asphalt matrix can be broken into pieces by hand but cores are still intact. Significantly aged and severe moisture related damage.	0.25	0.2
Very Poor	Surface cracks are throughout. Asphalt matrix is brittle and no intact cores. Very severe moisture related damage.	0.20	0.15
Very Poor	Extensive block cracks on AST surface.	0.20	0.15

**Used for preliminary design when FWD data is unavailable*

The total overlay SN is then converted to overly Bit. B thickness using the layer coefficient (a_1) of asphalt layer. No additional Bit. B is added as leveling on the milled surface. The mill depth may be increased and/or the overlay thickness may be further increased slightly to meet the lift thickness requirement. Table 14 presents examples for the calculation of overlay thickness for different rehabilitation options.

Pulverize Asphalt or Mill AST and Overlay: The SN loss (SN_{loss}) due to pulverization of existing asphalt or milling of AST surface are calculated as depth of pulverization or milling times the appropriate layer coefficient from Table 13. The pulverized asphalt is considered as an unbound granular material. The SN of the pulverized asphalt layer (SN_p) is calculated as the net thickness after relaying times its layer coefficient (0.14). The net thickness of milled and relayed AST is considered as zero. The net loss of $SN_{netloss}$ is calculated as the SN_{loss} minus SN_p. This $SN_{netloss}$ is added to the SN_{ol} to calculate the required total SN (SN_{tot}) of the overlay. SN_{tot} is then converted to asphalt or base plus asphalt layer thicknesses using their layer coefficients from Table 10. Additional bituminous (12.5 mm) is added as leveling requirements. The minimum bituminous thickness criteria must be met. Overlay thickness can be further increased slightly to meet the lift thickness requirement.

Cold in Place Recycle (CIR) or Full Depth Reclamation (FDR) and Overlay: The net SN loss ($SN_{netloss}$), total overlay SN (SN_{tot}) and overlay asphalt thickness are calculated in the same manner as the pulverization option described above. The CIR and FDR asphalt (treated with foamed asphalt or emulsion) layers are given layer coefficient values of 0.30 and 0.25 (these values are under review), respectively as opposed to 0.14 for pulverized asphalt. No additional bituminous is added as leveling requirements.

Table 14. Overlay design calculation for different treatments of existing surface.

PTH 10 (Region 3): Existing pavement thickness = 250 mm bituminous and 200 mm granular base; Condition of existing bituminous = fair; design traffic loading = 3.5 million ESALs; and subgrade = high plastic clay with a M_R of 26.2 MPa. SN_{dgn} = 165.1 mm; SN_{eff} = 108.2 mm; **SN_{ol} = 165.1 - 108.2 = 56.9 mm**				
Surface Layer Treatment	SN Loss Due to Treatment, mm	Required SN_{ol}, mm	Calculated Overlay Bit. B, mm	Total Bit. B, mm
No mill (Overlay)	0	56.9	56.9/0.40 + 12**	155
Mill 25 mm	25*0.30 = 7.5	64.4	64.4/0.40	161
Pulverize 250 mm	250*(0.30 – 0.14) = 40	96.9	96.9/0.40 +12**	255
FDR 250 mm	250*(0.30 – 0.25) = 12.5	69.4	69.4/0.40	174
CIR 100 mm	100*(0.30 – 0.30) = 0	56.9	56.9/0.40	142

***10-12 mm extra for levelling*

Design Example: Table 15 presents a comparison of overlay thickness for several projects in different regions of Manitoba. Overlay thickness reduction of 5 to 65 mm was possible using the new approach for designs provided since July 2017.

Table 15: Comparison of overlay thickness requirements for different projects.

Highway: Control Section	BBR Method	New Approach
PTH 15: 01 015 030HU (km 15.5-19.5)	165 mm Bit. B	115 mm Bit. B
PTH 3: 02 003 180HU (km 10.0-16.5)	115 mm Bit. B	85 mm Bit. B
PTH 2: 03 002 050HU (km 0.0-21.5)	140 mm Bit. B	135 mm Bit. B
PTH 23: 03 023 010HU (km 4.0-16.0)- 10 year	185 mm Bit. B	170 mm Bit. B
PTH 6: 04 006 060HU (Km 16.2-19.5)	110 mm Bit. B	95 mm Bit. B
PTH 39: 05 039 030HU (km 0.0-20.6)	200 mm Bit. B	135 mm Bit. B

Risk and Benefits of Changes to Pavement Design Process

There is always some risks associated with a change in any process, including the pavement deign approach. Risks associated with the design changes are: a) service life may be less than what used to be in the past; and b) may need earlier interventions than that used to be the past. However, historical analysis of pavement performance data showed that in general no pavement sections in Manitoba reached an international roughness index (IRI) value of 2.5 m/km within 20 years after construction.

The quantitative and qualitative benefits due to the changes are expected to outweigh the associated risks, which include:

1) No over investment on any project (save money);

2) Improve network health using saved money to other projects;
3) Technically more sound and consistent (less assumptions and no mixing of approaches); and
4) Use of new and more reliable technology (e.g., FWD data).

Concluding Remarks

Manitoba was using outdated and mixed approaches for pavement designs with inputs and adjustments that were not very sound and/or did not represent well local materials and conditions. As a more reliable design approach is still lacking, Manitoba re-explored the widely used AASHTO 1993 design method and undertaken major changes to its design practices. These changes resulted in a significant cost savings with more confidence in design. Further enhancements, verification of inputs and collection of data are in progress or planned to make the process more cost-effective and reliable, and in the long term adopt the AASHTOWare Pavement ME Design program.

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