Pavement Assessment & Design Manual (PADM) 2024

Manitoba Transportation and Infrastructure
Engineering and Technical Services - Highway Design Branch

Research & Innovation

Assessment

Materials

Design

Drainage

Reuse & Recycle

Sustainability & Climate Resiliency

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July 2024 (Interim Edition)

Manitoba

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FOREWORD

Manitoba last updated their pavement design manual in 2004 (MTGS 2004). A few changes to the pavement design practices were made in 2009 without a complete review of the 2004 manual. As the province expected to adopt the new Mechanistic-Empirical Design Guide (MEPDG) software AASHTOWare Pavement ME Design (NCHRP 2004), no further update to the design manual was undertaken since 2009. However, due to several limitations of the AASHTOWare Pavement ME Design (PMED) approach, Manitoba has been repeatedly delaying the full implementation of this new design tool. Although the first commercial version of the MEPDG or PMED software was released in 2007, the evaluation version was released in 2002. Despite the elapse of more than two decades since its first release, several critical issues are yet to be resolved to make this software widely acceptable. Therefore, Manitoba has postponed its full implementation until more development to the software and further evaluation, calibration and validation in Manitoba context.

This new edition of Manitoba's pavement design manual, named as the *"Pavement Assessment and Design Manual (PADM)"*, is a full rewrite of the manual to make it a more comprehensive and practical document. It reflects the current state of practices for pavement assessment and layer structure design for Manitoba provincial roads and highways. This new manual embraced tremendous changes in design approaches, practices and inputs as compared to those which were in use prior to 2018. One of the unique features of this new design manual is that it clearly states the rationale for each change in design process and input parameters, in addition to providing the step-by-step design procedures and design examples. Major changes to design process and inputs include, but are not limited to, the following:

1) Use tables from the AASHTO 1993 pavement design guide, instead of Modified Shell Equations, to calculate the axle load equivalency factors;
2) Replace Benkelman Beam Rebound (BBR) deflection with deflection basin measured using the Falling Weight Deflectometer (FWD) in designs for pavement rehabilitation and reconstruction;
3) Use of laboratory or field measured subgrade and pavement layer inputs, which represent materials that are currently in place or use in Manitoba;
4) Use of project specific heavy vehicle class distribution and traffic growth rate to determine the design traffic loads;

5) Establish and use a new set of serviceability indices considering the achievable construction quality, highway functional and strategic classifications, traffic volume and the desired level of services on different highways in Manitoba;

6) Implement a new set of design reliability levels considering traffic volume, surface type, highway functional and strategic classifications and highway context (urban, suburban, rural, rural town, park, remote, etc.);

7) Develop and implement a new approach for considering drainage qualities of different pavement layers/materials into the design;

8) Develop and recommend a new approach for considering frost susceptibility of subgrade soils into the design;

9) Develop and recommend a new procedure to determine the minimum pavement structure for a non-spring weight restricted highway with low traffic loads; and

10) Develop and use a procedure to determine the minimum pavement structure for a given project, which is required to carry traffic load over the seasonal shutdown period.

This new manual also provides an overview of pavement engineering principles including basics of pavement structures, pavement distresses and their feasible treatments, pavement materials and their characterization, pavement drainage, and the process to determine pavement design inputs. The complementary information is limited to providing a good understanding of the relevant concepts, issues and measures as this manual is meant to be a more practical design guide rather than a comprehensive textbook.

The methodologies presented in this manual apply to the design of gravel, asphalt surface treatment (AST) i.e., chip seal, asphalt concrete (AC) and portland cement concrete (PCC) surfaced, and composite (AC over PCC surfaced) pavement structures for new construction, reconstruction and rehabilitation on provincial highways and roads. The basic design methodologies are based on the 1993 AASHTO Guide for Design of Pavement Structures (AASHTO 1993). The design inputs and criteria for pavement design and analysis using the PMED software (AASHTO 2020) will be provided in a separate guide or manual once Manitoba is well satisfied with the outcome from this software.

The primary objectives of this manual are to assist pavement design professionals in assessing existing pavements and embankment conditions, assessing proposed new grades and providing the required (design) thickness for each layer of a pavement structure including recommendations for each layer material type and its treatment following a consistent approach. The manual reflects the most appropriate design methodologies, tailored for Manitoba conditions, and the current local experience and materials. Changes in technologies related to field testing and evaluation of pavement structures and materials, laboratory testing and analysis of materials, pavement response and performance equations (e.g., new empirical, mechanistic or mechanistic-empirical models), materials and construction specifications, the use of new materials as well as maintenance and preservation practices, changes in heavy vehicle configurations, axle combinations and allowable axle loads, climate change, etc. will influence the future performance of pavements. These will warrant changes to the design inputs in the future. Such changes to design inputs will be reflected in relevant engineering standards until a revision to this manual becomes desirable.

The manual is not all encompassing in terms of addressing all factors that may influence the design and performance of a pavement. Pavement design professionals will need to address those additional factors on a project-by-project basis and, where necessary, will have to carry out additional testing, research/assessment, and analysis to ensure that appropriate and cost-effective treatments and design solutions are provided.

DISCLAIMER

This design manual is meant to provide guidance on how to carry out pavement design, analysis and assessment for Manitoba highways and roads. The recommended inputs reflect the values based on the currently available data. Manitoba strives to continue research and development including adoption of new materials, specifications and technologies which may result in some changes to design input values. Such changes will be reflected in department's relevant engineering standards instead of resource intensive frequent revision to the design manual. The designer should check for the available latest version of the department's engineering standards, which may affect the design, analysis and assessment, when working on a Manitoba project.

This manual is not a formally copyrighted document or publication and it is open to use by any individuals, institutions, private entities and highway agencies. However, no part or content of this manual should be included in any other report, technical paper, guideline, manual, etc. without proper reference to this manual.

ACKNOWLEDGEMENTS

This new edition of pavement design manual has been prepared by Dr. M. (Mohammad) Alauddin Ahammed, P.Eng., the Senior Pavement Engineer and Manager of Pavement & Materials Engineering Section of Manitoba Department of Transportation and Infrastructure (MTI). Dr. Ahammed has also led the testing, data collection, analysis, research and performance evaluation of materials and pavements for the changes in design practices as well as design inputs and has developed an in-house pavement design and analysis tool. Several other staff of the department, as named below, have contributed by reviewing the manual, enhancing the in-house design tool, and by performing some investigation and analysis.

1) Dustin Booy, M.Eng., P.Eng., Executive Director, Highway Engineering;
2) Denise Jubenvill, P.Eng., Technical Services Engineer, Western Region;
3) Nicole Fleury, P.Eng., Technical Services Engineer, Capital Region;
4) Jeff Tallin, M.Sc., Eng., P. Eng., Senior Geotechnical Engineer, Highway Design Branch;
5) Yasir Shah, P.Eng., Pavement Design Engineer;
6) Marcus Wong, P.Eng., Pavement Assessment Engineer;
7) William Tang, P. Eng., Pavement Analysis Engineer;
8) Ryan Thompson, P.Eng., Pavement Asset Management Engineer;
9) Blair McMahon, Director, Environmental Services;
10) Warren Radbourne, Technical Services Engineer, Northern Region;
11) Katy Schram, P.Eng., Transportation Systems Planning Engineer;
12) Andre Dupuis, B.Sc., Surfacing Program Manager; and
13) Mumtaz Habib, P.Eng., Senior Highway Project Engineer, Capital Region.

David Mandrick, together with Andre Dupuis, coordinated the materials sample collection for laboratory testing and characterization, collected pavement surface and subsurface temperature data, and assembled pavement test sections data to support various analysis. The department's Mobile Operations and Central Laboratory staff provided great assistance by collecting samples of different materials, in-situ testing and data collection, laboratory testing, etc. Joyce

Marchand, Pavement Management Technologist, helped with the development of the cover page for this manual. Gerardo Parina, CAD Drafting Technician (Highway Design Branch) and Subhankar Ghosh, Design Technologist (Capital Region) helped with the development of several graphics for this manual.

The changes to design inputs, pre-engineering work and design methodologies were also agreed by the construction and technical services teams of the department. Two technical papers were published and presented at the 2018 TAC Annual Conference (Ahammed 2018-1 and Ahammed 2018-2) outlining the key changes that Manitoba have made to its design and assessment practices. These papers and presentations received an overwhelming acceptance.

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ABBREVIATIONS AND DEFINITIONS

AADT Annual Average Daily Traffic (AADT). The daily traffic volume on a highway section, which is averaged from an entire year of traffic counts on that section and reported as the total number of vehicles per day.

AADTT Annual Average Daily Truck Traffic (AADTT). The daily truck traffic (heavy vehicles) volume on a highway section, which is averaged from an entire year of truck traffic counts on that section and reported as the total number of trucks per day.

AC Asphalt Concrete (AC). A mixture of aggregate, asphalt cement and any other approved additives, which are mixed in a design proportion to meet specific properties.

AC Course Asphalt Concrete Course (locally called bituminous course). A layer of plant produced hot mixture of aggregate, asphalt cement and any other approved additives, mixed in a design proportion to meet specific properties, which is placed and compacted on a road and/or any other designated areas (e.g., parking lot, sidewalk, active transportation path).

AC Overlay Asphalt Concrete Overlay. One or more lifts of asphalt concrete mixture on an existing pavement. It may include a levelling course and one or more lifts of same or different asphalt mixture(s).

ACP Asphalt Concrete Pavement (ACP). A pavement structure surfaced with layer(s) of asphalt concrete mix(es).

ADT Average Daily Traffic (ADT). The daily traffic volume on a highway section, which is averaged or projected from a short period (usually 48 hours or less) of traffic counts on that section.

ADTT Average Daily Truck Traffic (ADTT). The daily truck traffic (heavy vehicles) volume on a highway section, which is averaged or projected from a short period (usually 48 hours or less) of truck traffic counts on that section.

Additive An agent added to soil or a pavement layer material to improve its physical, chemical and/or mechanical characteristics including constructability for the ease of placement and compaction.

ALS Axle Load Spectra (ALS). The distribution of weights from heavy vehicles on different axle groups (e.g., steer, single, tandem, tridem and quad axles) by classes of heavy vehicles. The distribution can vary on a monthly basis.

Analysis Period A specified period of time, which is used in an economic analysis to compare costs of alternative pavement surfacing or treatment options.

Asphalt Content The quantity of asphalt cement (asphalt binder) in an asphalt concrete mixture, expressed as a percentage of the total weight of the asphalt mixture.

AST Asphalt Surface Treatment (AST). A spray application of liquefied (e.g., emulsified) asphalt onto a road surface followed by the placement and rolling compaction of a thin layer of uniform or graded aggregates.

Axle Group One or more axles that are assembled together under an independent suspension as a single integral unit and attached to a vehicle chassis to distribute its weight to pavement surface. The common axle groups (axle units) are single steer, tandem steer, single, tandem and tridem. Quad axles are allowed in some jurisdictions, but they are not legal in Manitoba.

Axle Spread The longitudinal distance between the centres of outer axles (wheel sets) of an axle unit.

Axle Spacing The longitudinal distance between the centres of inner axles (wheel sets) of two adjacent axle units. It is also called the Inter-Axle Spacing.

Backcalculation It is a process of determining the structural capacity and layer moduli of an existing pavement from deflection basin data collected using a Falling Weight Deflectometer (FWD).

BBR Benkelman Beam Rebound (BBR)**.** The rebound deflection of a pavement structure, which is measured by applying a standard 40 kN static load on a set of dual tires of a single axle.

Break and Seat A technique of fracturing jointed reinforced concrete pavement. It ruptures the reinforcing steel across each crack or break and breaks its bonds with the surrounding concrete. The broken slabs are then compacted in place before overlaying with asphalt concrete or portland cement concrete surface layer.

CCP Compacted Concrete Pavement (CCP), also known as Roller Compacted Concrete Pavement (RCCP). A concrete (rigid) pavement with a very low or zero slump concrete mix, which is placed with an asphalt paver or grader and compacted in place with vibratory rollers.

Chip Seal An application of asphalt binder material followed by a cover coat of uniform or graded aggregates to any type of road or pavement surface. The thickness of a chip seal is usually 10 mm (single chip seal) to 20 mm (double chip seals).

CIR Cold In-place Recycling (CIR). A process of milling and reclaiming an existing AC layer to a specified depth, crushing to break (pulverize) the reclaimed asphalt pavement (RAP) into desired sizes, adding and mixing new asphalt binder (asphalt cement or emulsified asphalt) into the RAP, relaying to specified width and thickness, and finally, compacting to form a bound mat (layer). Corrective aggregates and a cementitious material may be added to improve volumetric and durability properties. Rejuvenating agents may be added to reactivate the binding properties of aged asphalt cement in RAP.

Cold Planning (Milling) A process of milling an existing asphalt pavement surface to a precisely controlled depth to remove bumps, ruts or deep cracks.

Complex Shear Modulus and Phase Angle Complex shear modulus (G*) is the total resistance to deformation of an asphalt binder sample when repeatedly sheared in Direct Shear Rheometer (DSR) test. The phase angle (δ) is the lag between the applied shear stress and the resulting shear strain. The larger the phase angle (δ), the more viscous the material is. G* and δ are used as predictors of asphalt concrete rutting and fatigue cracking.

Context Classification Context classification identifies surrounding land use: Rural, Rural Town, Suburban, Urban, Park/Culturally Sensitive, Remote and Winter Roads. Design inputs that are applicable to rural x-section (rural context) will apply to all highways/roads with full depth (≥900 mm deep) roadside drainage ditches. Design inputs that are applicable to semiurban x-section will apply to all highways/roads with medium depth (300 mm to <900 mm deep) roadside drainage ditches. Design inputs that are applicable to urban x-section will apply to all highways/road with shallow or no (<300 mm deep) roadside drainage ditches.

Crack and Seat A fracturing technique for jointed plain concrete pavement which involves cracking the slab into pieces, typically 300 to 900 mm in size. The PCC slabs are then compacted in place before being overlaid with a new PCC or AC course.

DL Design Lane (DL). The traffic lane or the travelled way of the road which is expected to carry the highest number of axle load repetitions among the lanes to be constructed or rehabilitated.

DLF or LF Design Lane Factor (DLF) or simply, Lane Factor (LF). The proportion of total trucks per day that are expected to use the design lane.

Design Service Life The number of years that a pavement structure should maintain an acceptable level of service (e.g., the terminal serviceability should remain at or above a selected level) without any additional structural rehabilitation intervention within that time frame.

Ditch Depth The depth of a roadside ditch measured from the embankment (subgrade) surface to ditch bottom.

Drainage Coefficient Factors used to modify structural layer coefficients of materials for flexible, semi-flexible and gravel surfaced pavements to account for the adverse effect of water infiltration on the stiffness (load carrying capacity) of unbound aggregate (base, subbase and fill) layers.

DF Drainage Factors (DF). Factors used to modify stresses in rigid pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration.

Dynamic Modulus (E*) The Dynamic (complex) modulus represents the structural response of a linear viscoelastic (LVE) material, such as the compacted hot mixed asphalt concrete, under loads at different temperatures and frequencies. It is a stress-to-strain relationship under a continuous sinusoidal load. It is used to determine the rutting and fatigue cracking performance of the asphalt mix.

ESAL Equivalent Single Axle Load (ESAL). ESAL of an axle group or unit is the number of equivalent load repetitions in terms of pavement damage caused by that axle in comparison to the damage caused by single pass of a standard 8,165 kg (80 kN or 18,000 lbs) single axle load.

FWD Falling Weight Deflectometer (FWD). An equipment that applies dynamic impulse loads, simulating the applied axle loads from moving trucks, and measures the resulting surface deflections (deflection basin) of a pavement structure with a series of geophones (sensors).

Grade Widening The construction of additional embankment to widen an existing highway while maintaining the existing road surface.

GBC Granular Base Course (GBC). A layer of untreated material of specified thickness placed below the AC, portland cement concrete (PCC), chip seal and granular surface layer or placed as a top layer on unpaved shoulders and gravel roads.

GSB Granular Subbase Course (GSB). A layer of untreated material of specified thickness placed below the granular or treated base layer.

GVW Gross Vehicle Weight (GVW). The total weight of a vehicle or combination of vehicles including its own weight and the loads that are being carried by the vehicle and are ultimately transmitted through its axles to the pavement as the applied stress.

GI **Group Index (GI).** An index number representing the relative properties and quality of different soils. It is a function of the liquid limit, plasticity index and the amount of material passing 75 um sieve.

Growth Rate The rate at which the traffic or truck volume is estimated to increase over a period of time.

Functional Classification Functional classification defines the role a highway plays in the overall highway network in terms of mobility and access. The classification is normally based on traffic volume, quality of connection between origin and destination, regional activities and the localized development or activities. The common functional classes are: Freeway, Expressway, Primary Arterial, Secondary Arterial and Collector. These classes dictate the basic geometric design of highways and roads.

Highway Loading Classification The highway loading classification **(**RTAC, A1, B1 and Residential) prescribes the allowable axle weights, gross vehicle weights and vehicle dimensions which are set out in the provincial regulation.

HIR Hot In-Place Recycling (HIR). A process in which a deteriorated/old AC layer is heated and scarified in place, mixed with a rejuvenator and/or new AC mixture, levelled and compacted to form a refreshed/recycled AC surface.

HMA Hot Mixed Asphalt (HMA), also called Hot Mixed Asphalt Concrete (HMAC) or simply Asphalt Concrete (AC). A plant produced hot mixture of aggregate, asphalt cement and any other approved additives, mixed in a design proportion to meet some specific properties, which is placed and compacted on a road and/or any other designated areas (e.g., parking lot, sidewalk, active transportation path).

Layer The total thickness of a particular component of a pavement structure with no change in material properties (e.g., AC material) which is placed in one or more lifts.

Levelling Course A layer of AC mixture placed on an existing surface to restore or improve its cross-fall and profile.

Lift The compacted thickness of a pavement material or subgrade laid in a single application.

LL Liquid Limit (LL). The water content, expressed as a percentage of the weight of the oven-dry soil, at which a soil passes from a plastic to a liquid state.

p_i Initial Pavement Serviceability Index (p_i). The serviceability index of a pavement surface that can be achieved after new construction, reconstruction or rehabilitation of that pavement structure.

Physical Property The inherent attributes or features of a material (subgrade soil, aggregate, asphalt mixture, asphalt binder and portland cement concrete mixture).

p_t Terminal Pavement Serviceability Index (p_t). The lowest level of serviceability index for a pavement surface that will be acceptable before resurfacing, rehabilitation or reconstruction becomes necessary.

PL Plastic Limit (PL). The lowest water content, expressed as a percentage of the weight of the oven-dry soil, at which a soil remains plastic and it changes from a plastic to a semisolid state below that water content level.

PI Plasticity Index (PI). The numerical difference between the liquid limit and the plastic limit of a soil or aggregate material.

Poisson's Ratio Poisson's ratio is defined as the ratio of the lateral strain to the axial strain due to the application of an axial load.

PCC Portland Cement Concrete (PCC). A mixture of blended aggregates and portland cement paste with or without addition of chemical and other additives. The cement paste binds the aggregates into a rocklike mass as the paste hardens due to the chemical reaction between cement and water. Additives are usually added to reduce water content, increase workability, enhance durability, provide adequate entrained air voids, and accelerate or retard the setting time.

RAP Reclaimed Asphalt Pavement (RAP). The AC layer of an existing pavement that has been removed and processed for the purpose of recycling in a new hot mixed AC mixture, cold in place recycling or reusing in any other form.

Recycling The process of reclaiming an existing pavement material, reprocessing it with or without additional material, additive or binder, and relaying on the roads.

Resilient Modulus It is the ratio of the applied cyclic stress to the recoverable (elastic) strain under cycles of repeated loads. Thus, it is a direct measure of stiffness for unbound materials (e.g., granular base, subbase, fill and subgrade) in pavement system.

Rubblizing A process in which a PCC pavement is crushed and broken by vibratory or mechanical action into sizes of 50 to 150 mm, and , where applies, the bond between steel and concrete is shattered. The crushed and broken concrete is then compacted to form the base material for a new surface layer. Additional granular material can be placed on rubblized concrete layer prior to the placement of surface layer.

Rural x-Section Roadway x-section with a roadside ditch of ≥900 mm depth, measured from the embankment (subgrade) surface to ditch bottom.

Sandwich Course A layer of granular or treated material of specified thickness placed in between an existing pavement surface and a new AC or PCC surface or between two new bound material layers.

Saw and Seal A process which attempts to control reflective cracks in an asphalt concrete overlay on PCC pavement. The asphalt concrete overlay is sawed (routed) directly over the PCC transverse joints prior to sealer application.

Semiurban x-Section Roadway x-section with roadside ditches of ≥300 mm to <900 mm depth, measured from the embankment (subgrade) surface to ditch bottom.

Service Life The period of time that a newly constructed, rehabilitated or reconstructed pavement will actually perform before reaching its terminal serviceability.

Single Axle An axle unit with only one axle under an independent suspension.

Stabilization The addition of a binder material such as lime, portland cement, asphalt cement or emulsion to an unbound material such as soil and aggregate including reclaimed (and processed) asphalt pavement to transform it into a bound or semi-bound layer that increase its stiffness or load carrying capacity.

The modification of a primary material, e.g., soil, aggregate (including RAP) and asphalt concrete, is different from the stabilization. In the modification process, a small amount of modifier such as portland cement, lime, emulsified asphalt and/or chemical additive(s) are incorporated to alter some properties or condition of a primary material. It does not convert a primary material type into a different primary material type and/or results in a considerable increase in stiffness or structural value of the primary material. For example, the addition of a small amount of cement or lime to alter the plastic characteristics of a soil that reduces its swelling and shrinkage potential or reduce moisture content that makes the soil workable and expedites the construction; the addition of a small amount of emulsion to aggregate, reclaimed or pulverized asphalt material to enhance its workability and to provide a good workmanship of a temporary riding surface.

Steering Axle The lead axle unit (single or tandem axles) of a vehicle which governs the direction of travel of the vehicle.

Strategic Classification Strategic Classification identifies routes that serve a strategic role provincially, especially with respect to economic enablement: Trade, Commerce, Commuter, and Recreational.

Surface Course The top layer of a pavement structure.

Surface Smoothness Longitudinal profile of the pavement surface, measured with a profiler and expressed as International Roughness Index (IRI). Manitoba uses high-speed inertial profiler.

Tandem Axle An axle unit with two consecutive axles under a single suspension having an axle spread of not less than 1.0 m nor more than 1.85 m.

TCP Thin Concrete Pavement (TCP). A PCC pavement with short panels and no load transfer dowels, which is designed to reduce PCC slab thickness based on the optimization of tensile stresses induced due to applied wheel loads, load repetitions and slab curling.

Tridem Axle An axle unit with three equally spaced consecutive axles under a single suspension, having an axle spread of not less than 2.4 m nor more than 3.7 m.

TEF Truck Equivalent Factor (TEF). The number of 8,165 kg equivalent single axle load repetitions per truck of a mixed truck traffic stream on a highway section.

TF Truck Factor (TF). The number of 8,165 kg equivalent single axle load repetitions per truck of each heavy vehicle configuration.

Urban x-Section Roadway x-section with roadside ditches of <300 mm depth, measured from the embankment (subgrade) surface to ditch bottom.

Chapter 1: INTRODUCTION

1.1 Overview

Among the different forms of transportation, road transportation has been the principal mode for the movement of people, goods and services throughout Canada. Trucking, which generally moves food products and manufactured as well as processed goods via road network, is the principal method of intra-provincial, inter-provincial and international freight haul in Canada (Transport Canada 2018). Given that trucking is the dominant method of transporting goods and services, the provision of sound and safe pavement structures and surfaces on the road network is one of the most important aspects of road infrastructure design and management.

The cost of pavement structures constitutes major part of the total costs of all highway construction and rehabilitation projects. A reduction in pavement structure thickness corresponds to a reduction in construction cost and contributes to better management of road network health. However, such reduction of thickness may result in structurally inadequate pavement and reduction in service life or life cycle. Therefore, it is important to optimize pavement structure thickness to achieve the desired performance or service life without overspending on any project. To make highway construction more cost-effective, the department re-explored the widely accepted/used AASHTO 1993 Guide (AASHTO 1993) approach and had undertaken major revisions to its design practices. Changes from the previous design manual include, but not limited to, the following:

1) Provide an overview of pavement engineering principles including the different design methods, pavement distresses/failures and treatments/interventions, life cycle cost analysis, pavement materials and the sustainability and climate change consideration in pavement design and material selection;
2) Use project specific Truck Equivalent Factor (TEF) value that varies based on truck traffic class distribution, highway loading class and pavement type;
3) Calculate the ESALs for each axle type based on Axle Load Equivalency (ALF) factors presented in AASHTO 1993 guide instead of the Modified Shell Method;
4) Use a new process for a more reasonable estimate of project specific truck traffic volume, instead of selecting the truck volume directly from a short-term count, and its growth rate;

5) Use FWD central deflection to determine existing pavement structural capacity and discontinue the Benkelman Beam Rebound (BBR) deflection method in the design for pavement rehabilitation;

6) Use of FWD deflection basin to determine subgrade stiffness for rehabilitation and reconstruction projects instead of soil group index;

7) Use effective subgrade moduli considering seasonal variation subgrade stiffness and pavement drainage conditions;

8) Use new engineering principles to guide the design of pavement structures on frost susceptible subgrade soils and to manage frost heave issues based on field experience of frost severity, frequency and extent, and the composition of subgrade material and expected frost heave rate;

9) Emphasize the use of resilient modulus of subgrade soils containing organics, instead of generic correction to structural number, whenever possible;

10) Revise the structural layer coefficients of AC materials, based on laboratory testing, to more closely represent local materials that are currently in use;

11) Determine and use the resilient moduli of local unbound base and subbase materials through laboratory and field testing and use the annual equivalent moduli and structural layer coefficients of these materials considering drainage and seasonal conditions.

12) Recommend a new set of design reliabilities considering the importance of each highway section in terms of traffic volume, surface type, highway functional and strategic classifications and project context i.e., cross-section (x-section) type;

13) Select appropriate initial pavement serviceability index (P_i) values based on local construction quality (initial smoothness), which varies depending on the pavement surface type and thickness;

14) Recommend a new set of terminal serviceability index values considering the importance (i.e., the desired or manageable level of service) of each highway section in terms of highway functional and strategic (e.g., trade, commerce, commuter and recreational) classes, highway/road contexts (e.g., urban, semi-urban, rural) and traffic volume categories;

15) Incorporate layered design analysis to determine the minimum thickness of the surface and base layers;

16) Develop and use new design criteria for determining the minimum pavement structure for non-spring weight restricted highways with low traffic loads;

17) Develop and use a process to determine the minimum pavement structure, which is required to be placed prior to the seasonal shutdown of construction;

18) Incorporate a process to determine the project specific asphalt binder grade including the allowable contents of reclaimed asphalt pavement (RAP);

19) Develop a guideline for proper pre-engineering activities prior to issuing intermediate pavement designs, which can be used for construction;

20) Incorporate design procedure for PCC overlays; and

21) Incorporate design procedures for low volume and gravel roads.

In general, this manual reflects Manitoba's new design practices, which are expected to provide cost-effective and sustainable pavement structures with desired performance under Manitoba's environmental, traffic and materials conditions. Manitoba designs flexible, semi-flexible and gravel surfaced pavement structures for 20 years of initial service life. Rigid and composite pavement structures are designed for 25 years of initial service life. However, these pavements are expected to pass a 50-year life cycle at the desired service conditions, with the application of routine maintenance and planned preservation treatments, until major rehabilitation or reconstruction becomes necessary. The department places a high priority on the ride quality and serviceability of pavements, especially on major highways.

1.2 Background

Over the past several decades, the department had been using the BBR deflection method in the design for rehabilitation of existing asphalt concrete (AC) and AST pavements. The department had been collecting BBR deflection data during the spring season of each year until 2007. BBR data collected in spring represents the weakest condition of pavements within a year. Since the spring thawing and weak pavement condition last for about two months in each year, BBR data collected in spring does not represent the annual average condition of pavement structures and subgrade soils. The year-to-year variation of spring condition was also a major issue with the BBR data, in addition to its poor repeatability. As a result, the department discontinued the use of BBR deflection data in 2016 and started to use the falling weight deflectometer (FWD) data.

The department uses the AASHTO 1993 design guide approach to calculate the total structural number (SN) in designs for the new construction of flexible, semi-flexible (AST) and gravel road pavements and the rehabilitation and reconstruction of composite and rigid pavements with new AC surface. The thicknesses of composite (AC over PCC) and rigid (PCC) pavements are also determined using the AASHTO 1993 design guide approach. However, the department had been using the Modified Shell Equations to calculate the design traffic loads (in terms of Equivalent Single Axle Loads or ESALs), which produces higher design ESALs than that produced using tables provided in the AASHTO 1993 design guide. Therefore, the department discontinued the use of Modified Shell Equations in 2016.

The estimated values of subgrade resilient moduli and pavement structural layer coefficients that were in use in the designs were higher than the currently measured values. The specifications of pavement materials and construction have also been recently changed. Several adjustment factors were applied to the calculated design structural number to account for the subgrade soil frost susceptibility, organic contents and saturation, and the highway context (e.g., urban, semiurban and rural x-sections) based on professional judgement. Those factors are now accounted for in the designs differently based on measured design input parameters and/or newer engineering principles.

Although Manitoba is one of the leading agencies in Canada in terms of evaluating the AASHTOWare Pavement ME Design (PMED) approach, the department slowed its implementation due to several major issues that are yet to be resolved. The issues include, but are not limited to:

1) Low sensitivity of the predicted distresses and AC thickness design to subgrade stiffness;
2) Low sensitivity of the predicted distresses and thickness design to unbound granular material layers, especially for the increased thickness of granular layer(s);
3) Low sensitivity of the predicted thermal cracking due to the variation of climatic conditions (e.g., same amount of thermal cracking with PG 58-34 asphalt binder for all climatic conditions across Canada);
4) Low sensitivity of the predicted distresses to increased traffic loads, especially for rigid pavements;
5) Inconsistent variation of the predicted distresses due to changes in some design inputs, including subgrade type (for both flexible and rigid pavements);

6) A significant amount of predicted rutting in subgrade and granular base (and subbase) layer(s) after asphalt overlay of an existing flexible pavement despite no rutting in the existing pavement subgrade and granular base (and subbase) layer(s);

7) Significant differences and inconsistencies in the predicted distress between software versions (e.g., v2.6 versus v3.0); and

8) Need for calibration of transfer functions and distress prediction models to suit local materials, pavement structures, traffic loads, environmental conditions and observed distresses. However, the calibration effort on software models and/or transfer functions with many inconsistencies or issues is not likely to yield any beneficial result at this time.

Given these issues in the AASHTOWare PMED approach, the department will continue to the use of the AASHTO 1993 Guide approach until these issues are resolved.

The changes in design approach, which are presented in this manual, are intended to provide more accurate design of pavement structures on Manitoba provincial highways/roads.

1.3 Objectives

The primary objectives of this manual are to assist the pavement designers in the following key aspects:

i) Assessing the existing pavement structures for their load carrying capacity;

ii) Determining the most suitable rehabilitation or reconstruction strategies;

iii) Providing recommendations for pavement layer materials;

iv) Providing pavement layer designs using appropriate design inputs and process; and

v) Following a uniform approach in materials and pavement assessment as well as designs for durable and cost-effective pavement structures.

1.4 Manual Organization

This chapter (Chapter 1) provided a general introduction to the Manual. Chapter 2 includes basic concepts of pavements and their structural designs, pavement distresses and failures, pavement preservation and rehabilitation treatments, pavement drainage, surface type selection and the sustainability and climate change consideration. Chapter 3 provides a discussion of pavement materials including pre-engineering investigation and testing, data collection and pavement assessment. Chapter 4 provides details of traffic data analysis and calculation of design traffic loads. Chapter 5 provides details of subgrade soil stiffness and design inputs. Chapter 6 provides the design methodologies for flexible and semi-flexible pavements new construction and full depth reconstruction projects while Chapter 7 provides the rehabilitation and partial depth reconstruction design procedures for these pavements. Chapter 8 provides the methodologies for the design of rigid and composite pavements for new construction and reconstruction projects while Chapter 9 provides the methodologies for rehabilitation and partial depth reconstruction design of these pavements. Chapter 10 provides the design methodologies for new construction and reconstruction of gravel surfaced roads. The literature references are listed in Chapter 11. Pavement analysis and design using the AASHTOWare PMED Software will be covered in a separate manual. Life cycle cost analysis for the selection of pavement options and rehabilitation alternatives will be covered in a separate manual or guideline. The pavement preservation selection criteria and timing of each treatment will be covered in a separate guideline.

Chapter 2: BASICS OF PAVEMENT STRUCTURE

2.1 Overview

Pavements are horizontal structures of engineered materials and constructed on prepared subgrade soils to carry design traffic loads on roadways. Subgrade is termed as the foundation of the overlying pavement structure. A pavement structure must be sufficiently stiff and thick to distribute the imposed traffic loads over a wide enough area to limit the stresses on the subgrade. In addition, pavements are generally layered structures consisting of several material layers such as AC and/or PCC, granular base and granular subbase. Each underlying layer acts as the foundation for the overlying layer(s) and each layer undergoes traffic and environment related stresses. Therefore, each layer of a pavement must be sufficiently stiff and thick to avoid overstressing the underlying layer and subgrade. Satisfying these conditions will eliminate the potential subgrade and overall pavement failures (deformation, settlement, shear, etc.) and avoid premature surface and layer distresses. In addition, the better the quality of the materials and their placement, the better the performance of a pavement. To provide an appropriate design of pavement structure for a highway section, a sound knowledge of material properties and their impact on pavement performance is critical.

The intensity of the induced stress due to an imposed traffic load is maximum at the pavement surface. The stress intensity reduces with increased depth, as an applied load is distributed over a wider area with increased depth, from the pavement surface to the subgrade (see Figure 2.0.1 as an example). The materials at and near the pavement surface are also more exposed to the natural environment and changes in climatic or weather conditions. Accordingly, highly durable materials, which are strong and exhibit high resistance to wear and disintegration, are placed at and near the top surface. Lower quality (i.e., weaker, less stiff) materials can be placed in deeper layers of a pavement structure where stress intensities are lower.

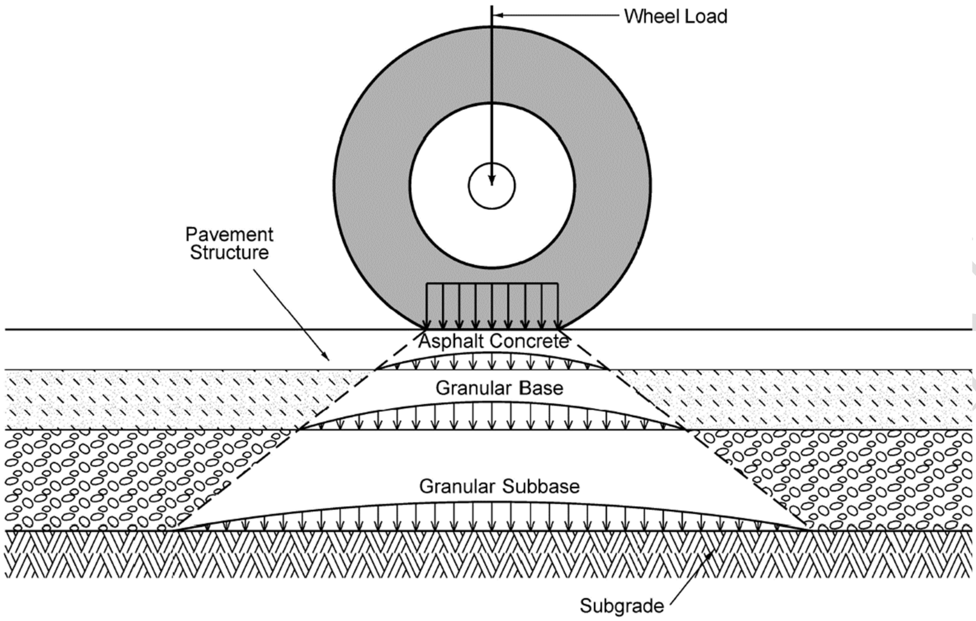

Figure 2.0.1: Distribution of Wheel Load through Pavement Structure (not to scale) (Adopted from Asphalt Institute 2007)

The U.S. FHWA Policy Guide (FHWA 1999) states that pavement structures should 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 to when designing a pavement include traffic loads, materials, climate, drainage, construction practices, and desired performance over the design service life. As pavements are built to facilitate traffic movement, an accurate estimate of traffic loads over the design service life is extremely important. The design traffic loads should represent the current truck volume, classification, weight and growth over the design service life.

Since the stresses from imposed traffic loads are ultimately transferred to the subgrade soil, the provision of a uniform and stiff foundation that can eliminate potential differential movement (e.g., deformation, settlement or expansion) of pavement and subgrade, and withstand potential damage due to frost and changing moisture is one of the key considerations in pavement design and construction. Granular and/or treated materials of adequate thickness should be used to provide a stable and uniform support to each pavement structure. Non-frost susceptible and free draining granular base and subbase materials of adequate thickness should be used in cold climates where pavements are exposed to frost and repeated freezing and thawing. Base and subbase materials should be resistant to degradation and changes in mechanical properties i.e., reduction of strength and stiffness due to stresses from imposed traffic loads, environmental exposure and changes in moisture content. The provision for adequate drainage and accounting for pavement structure drainage properties/conditions in the designs are also important factors to ensure the desired pavement performance (FHWA 1999).

For a rehabilitation design, it is essential that each project be properly engineered to ensure the most feasible and cost-effective option is chosen. 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 (e.g., physical properties). The selected rehabilitation treatment should address the observed distress and its reason to prevent premature reoccurrence (FHWA 1999).

2.2 Pavement Types and Uses

In general, pavement structures are categorized into different types based on their surfacing materials. Accordingly, pavements on Manitoba highways are grouped into five different types:

1) Flexible (asphalt) pavement: Asphalt concrete (AC) surfaced pavement structure, locally known as bituminous pavement;
2) Semi-flexible pavement: Asphalt surface treatment (AST) i.e., chip seal surfaced pavement structure;
3) Rigid (concrete) pavement: Portland cement concrete (PCC) surfaced pavement structure;
4) Composite pavement: Pavement structure with composite (AC over PCC) surfacing layers; and
5) Gravel roads: Granular aggregate surfaced pavements.

The top layer of a flexible pavement is typically composed of hot mix asphalt (HMA) (also called hot mixed asphalt concrete or hot mixed bituminous). Some agencies use a HMA wearing course, on the top of the main load bearing HMA course. The load bearing HMA layer is called the asphalt binder course. Regardless of whether two or more HMA courses are used or not, the surface course or layer must be designed to resist the forces of traffic, be waterproof to protect the lower layers from weakening due to moisture ingress, and provide a skid-resistant and smooth ride (TAC 2013). The base and subbase courses of pavement structures are generally composed of crushed or pit run granular material with the subbase containing lower quality material than the base. The base and subbase should be drainable to facilitate layer drainage.

The semi-flexible i.e., AST surfaced pavements consist of double chip seals applied on a layer of granular aggregate or directly on the compacted subgrade soil surface.

Rigid pavements consist of portland cement concrete (PCC) surface layer placed on a prepared subbase. Although rigid pavements generally do not require a base or subbase for structural support, base and subbase aid in minimizing pavement damage due to frost action, preventing squeezing of underlying subgrade soils due to repeated traffic loads and thereby, preventing slab faulting, improving drainage, minimizing erosion, shrinkage and swelling, and act as a working platform during construction. Manitoba typically constructs rigid pavements on weak (high plastic clay), frost susceptible and swelling subgrade soils. Therefore, the provision of a good quality granular base and subbase layers of adequate thickness to provide strong, stiff, drainable and stable foundation support is important for rigid pavement design and construction in Manitoba. Alternatively, a thin base layer (200-300 mm thick, depending on the subgrade soil type, contents and stiffness) could be placed provided that the top 300 mm of the subgrade soils below the granular base layer is stabilized with portland cement. For highway embankments subjected to erosion or washouts, cement stabilized subgrade and/or subbase should be considered for placing below a granular or treated drainable base layer, depending on the site condition.

A composite pavement is basically a rigid pavement with an additional overlying layer of AC on the top surface. On Manitoba highways, a composite pavement is usually formed due to the placement of an AC overlay on the PCC pavement surface that experienced roughness and faulting issues. A new composite pavement is typically constructed to match the new surface with the adjacent existing pavement surface.

Gravel (granular aggregate) surfaced pavements are usually constructed on low volume roads or access roads. They consist of a layer (75-100 mm thick) surfacing aggregate placed over compacted subgrade soil or another layer of granular material.

Perpetual (full-depth and deep-strength), also called long-life, pavements were great innovations in the 1960's, but they are not well known or commonly used. They are AC pavements designed and constructed to last 50 years or longer without requiring any major structural rehabilitation or reconstruction. They are designed to eliminate bottom-up fatigue (alligator) cracking and to withstand rutting due to traffic loads, and therefore, they require only periodic surface renewal (mill and fill) to remedy the surface distresses, which are confined to a thin wearing (surface) course. Some full-depth AC (placed directly on unmodified or modified subgrade soils) and deep-strength AC (placed on thin layer(s) of granular base/subbase) pavements have been constructed since the 1960's. The properly designed and constructed perpetual pavements have successfully provided long service lives under heavy traffic loads.. Compared to conventional AC pavements, which are typically consist of relatively thin AC and thick granular base/subbase layer(s), perpetual pavements are overall thinner, but consist of thicker AC layer on little or no granular base/subbase layer(s) (APA 2002).

A perpetual pavement is designed for durability and long service life with a rut and wear resistant AC surface layer, a rut resistant intermediate AC layer and a fatigue resistant AC base layer (APA 2002). Figure 2.0.2 shows a concept of the perpetual pavement structure. With proper design of pavement, selection of appropriate layer materials and good construction practice, a long-lasting pavement with low maintenance and preservation interventions can be achieved. Thus, this type of pavement structure may provide a cost-effective pavement strategy for highways with high traffic loads and also reduce traffic disruptions related to preservation and maintenance activities. However, the feasibility of this type of pavement, including its cost-effectiveness, in Manitoba's local subgrade soils and environmental conditions must be further investigated before making any decision for its construction.

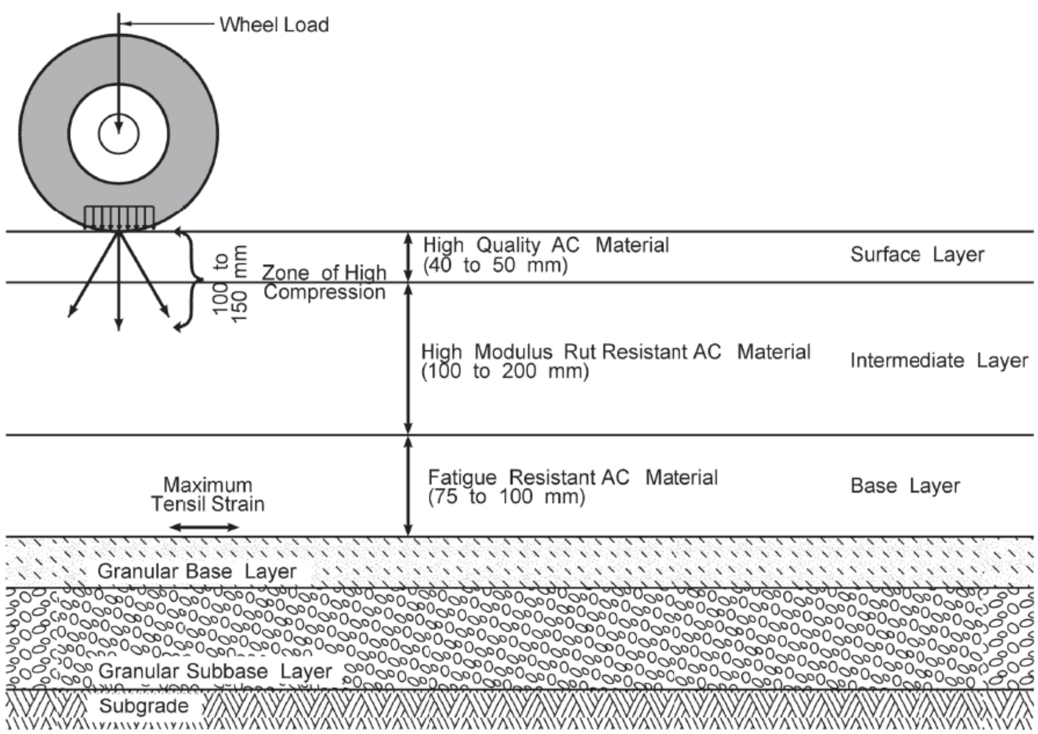

Figure 2.0.2: Perpetual Pavement Design Concept (Adopted from APA 2002)

Several alternative pavement types are also in use in some jurisdictions, which include the following:

1) Inverted pavement;
2) Semi-rigid pavement;
3) Permeable concrete pavement;
4) Porous asphalt pavement;
5) Compacted (or roller compacted) concrete pavement; and
6) Thin concrete pavement (TCP).

An inverted pavement consists of a stiff cement-treated layer (200 mm to 300 mm thick) placed on a compacted subgrade. An unbound granular base layer (150 mm to 250 mm thick) is then placed on the cement-treated layer. Finally, a thin (75 mm to 100 mm thick) HMA is placed as the surface layer. The unbound granular material inter-layer in an inverted pavement (also called inverted base pavement) plays a major role in the mechanical response of the pavement structure. An inverted pavement construction may costs 25% less than the construction of a conventional AC pavement (TRB 2016). However, further research and investigation are required before Manitoba can try such pavement structures to ensure that they are well suited with the local climatic conditions.

In 2015, Manitoba placed a trial pavement section on PR 330 (from PR 205 to the North Boundary of Regional Municipality of Morris) that resembles an inverted pavement. In this trial section, a 200 mm thick layer of cement-treated Granular A base was placed on the pre-existing gravel road surface. The cement treated base layer was overlaid with a 100 mm thick layer of untreated Granular A base, which was surfaced with double chip seals (AST). As the surface was AST, instead of AC, the stabilized base layer might have experienced higher stress from traffic loads than that in a standard inverted pavement. So far, the pavement is performing well, however pavement deflection was shown to increase (strength reduced) after exposure to traffic for three years indicating the development of some traffic and environment related distress (fracture) in the cement stabilized layer.

A semi-rigid pavement consists of HMA top layer placed over a cementitious stabilized material. Cementitious materials may include lime, lime-fly ash and portland cement stabilizers (MEPDG 2020). Technically, semi-rigid pavement is a variation of composite pavement and it is not yet constructed on any Manitoba highways.

Permeable concrete pavements are constructed with permeable paver blocks as surface layer (to allow for drainage through the surface), which are underlain by a cement stabilized permeable base layer. These permeable layers provide quick drainage of water from pavement surface. The permeable base layer also acts as a temporary reservoir for surface water. The current application of permeable concrete is limited to parking lots, low volume residential roads/streets, local road shoulders and low traffic bus lane. There is no application of this pavement type on main routes of highways due to: a) concern over the durability of the mix, b) difficulty to attain the desired surface smoothness; 3) issues with mix production and delivery; 4) higher cost than traditional pavements; 5) the requirement to follow a stringent construction method; and 6) the requirements for frequent and careful maintenance.

Porous asphalt pavements are not compatible with harsh winters. The de-icing salts and sands cause clog of the air voids in the pavement, which diminishes the drainage properties. In addition, permeable asphalt mixes with the required tensile strength for cold climates have not been developed yet. Further research is needed before a porous asphalt pavement can be used on Manitoba highways.

Compacted concrete pavement (CCP) or roller compacted concrete pavement (RCCP) are constructed with very low or zero slump portland cement concrete (PCC) mixes. CCP and RCCP concrete mixes contain about the same amount of cementitious materials as conventional PCC mixes. However, CCP and RCCP concrete mixes have lower water contents than conventional PCC mixes. The water content in the CCP and RCCP concrete mixes need to be sufficient only for compaction and hydration of cement. They are placed with a paver (typically, an asphalt paver) or grader and then compacted with vibratory rollers. The use of CCP and RCCP are so far mostly limited to heavy industrial facilities with some trials on intersections and municipal roads.

Thin concrete pavement (TCP) is a short slab (typically 1.8 m x 1.8 m panels) PCC pavement with no load transfer dowels at joints. A thinner design is achieved by optimizing panel size based on tensile stress induced due to truck wheel loads, design service life load repetitions and tensile stress due to slab curling. The slab geometry is optimized so that only one set of wheels (one half of each axle load) lands on a single slab at a time. The panel sizes in TCP are way smaller than the conventional PCC pavements. These significantly reduce the tensile stresses in PCC slabs allowing a thinner PCC pavement construction than the conventional PCC pavements. For example, 150 mm (6 inches) thick PCC slab on granular base was shown to be adequate for 12 million ESALs, while 200 mm (8 inches) PCC slab on granular base was shown to be adequate for 51 million ESALs in terms of fatigue cracking in both designs. However, these slab systems have experienced higher deflections than the conventional PCC pavements. Therefore, the granular base layer and subgrade must be designed, together with the provision of adequate drainage, to reduce permanent deformation and minimize the possibility of pumping and erosion (Cervantes and Roesler 2009). The swelling and frost susceptible subgrade soils in cold climates, like Manitoba, may cause performance and maintenance issues for TCP, and it may not be cost effective if a thick base layer is required.

2.3 Basics of Pavement Structural Design

The basic approach of a pavement structural design, using any design methodology, is the selection or determination of the appropriate layer thickness of available or intended materials based on the following key design parameters:

- traffic loads,
- subgrade type or stiffness/strength,
- layer material stiffness, and
- the selected design service life.

Additional considerations include the following:

- local environmental and drainage conditions,
- seasonal variation of moisture and temperature,
- pavement surface type,
- subgrade soils frost and swelling issues,
- organics and peat issues,
- consistency of layer materials and surface types with the layer materials and surface types of adjacent road sections or areas,
- constructability, and
- any special treatment of a layer material such as cement stabilization of soil or granular (aggregate) material.

For a rehabilitation design, additional consideration is the selection of appropriate treatment type for the existing pavement surface layer based on the type, severity and extent of the observed pavement distresses in the existing pavement. A good understanding of the pavement distresses, their causes and suitable treatments to rectify or minimize recurrence of the observed distresses in the remaining/treated existing pavement and/or reflection to the newly paved surface within the desired service life is critical to rehabilitation design.

The pre-selected terminal service quality at the end of the design service life is an important factor in both empirical and mechanistic-empirical design approaches. The terminal service quality index could be in terms of a composite index such as the present serviceability index (terminal), a criterion used in the AASHTO 1993 method, or individual pavement distresses such as the acceptable roughness, amount of cracks, rut depth and faulting that are used in the AASHTOWare PMED approach. The pavement maintenance and preservation treatments that are applied to the pavement surface to extend the service life beyond the design service life are usually ignored in traditional pavement structural design. However, the AASHTOWare PMED (MEPDG 2020) has incorporated an option to consider one of the typical pavement preservation (non-structural maintenance) treatments that are applied within the selected design service life to target a high rutting issue. The treatment applied to address rutting can reduce other distresses to a limited extent depending on the selected treatment type, which is ignored in the design using the PMED software. However, this option provides an avenue for structural design for a longer service life where rutting is the predominant pavement distress, once the PMED software could be implemented.

The overall quality of construction plays a significant role in the actual service life of pavements. No design will work if it is not applied i.e., constructed to the pre-specified quality. The attainable construction quality is therefore an important consideration in this newest design manual of Manitoba.

Finally, the selection of pavement surface type, layer material(s) type and existing layer treatment is often driven by the initial costs, available funds and project schedule, although the most appropriate alternative option(s) could provide savings in terms of lower life cycle costs and reduce the future maintenance/preservation activities, resource needs and traffic disruptions. Highway right-of-way (ROW), elevation (e.g., bridge clearance, rail tracks) and other constraints may dictate alternative options with a reduced design service life or full depth reconstruction. Pavement designers should consider those constructability limitations when recommending pavement structure option(s) including surface type(s), existing pavement treatment(s) and layer material(s). Sustainability and climate change should also be included into the decision matrix.

2.4 Pavement Design Methodologies

Several design approaches have been used and are still in use globally. The principal concepts or methodologies underlying all design approaches are:

1) Experience-based;
2) Empirical;
3) Mechanistic; and
4) Mechanistic-Empirical.

2.4.1 Experience-based Design and Standard Sections

The experience-based method involves the selection of a standard (typical) pavement structure (called standard section) for a new project based on the performance of similar projects that were successfully completed in the past for similar condition of traffic loads, subgrade material type, layer materials and thickness as of the new project. Some agencies have developed and used tables of experienced based standard sections for the new construction of pavements for its simplicity in use. The pavement rehabilitation practice has generally been carried forward through knowledge transfer to new practitioners or regional maintenance staff. Examples of experienced based design in Manitoba are typical base thickness for gravel and AST surfaced pavements as well as the typical subbase/base and PCC layer thicknesses for rigid pavements.

A newer approach toward the use of standard sections is called design catalogue where an agency uses empirical, mechanistic and/or mechanistic-empirical pavement design method(s) to determine the pavement layer thickness (including material types) for different traffic levels (may include highway functional class as a variable) and subgrade types within each jurisdiction. Then the most appropriate layer materials and thickness are selected from the design catalogue to recommend pavement structure for each specific new project situation.

The experience-based method or the newly developed standard catalogue-based sections may be used where a new project condition closely matches with a previously used design or design inputs, given that no other variables exist at the new project location. However, changes in climate, material specifications and properties, construction specifications, pavement maintenance and preservation practices, and truck traffic axle loads may make these methods unsuitable for most cases. Furthermore, variation in subgrade soil contents such as moisture and organics, subgrade stiffness (within a particular soil type/class), soil frost and swelling susceptibility, variation in material sources and the quality of aggregates for pavement layer materials and drainage condition could make the standard designs unsuitable or inappropriate with under design (early failure) or overdesign. Both scenarios have cost implications. As such, the use of these methods, if chosen, should be restricted to gravel, AST and thin AC surfaced roads with a low traffic volume or loading conditions, where a higher risk is tolerable as the consequence of early failure is not enormous.

2.4.2 Empirical Design Method

Empirical design approaches are developed based on correlations between the design inputs and the observed field performance. As these design approaches are based on field data, they are more accurate and flexible than the experience-based design, and they are not very complicated to use. However, the validity of these approaches may be limited to the boundaries of input data that were originally used to develop the empirical correlations. It may not be possible to readily incorporate inputs for new materials, the impact of changes in construction procedures, traffic loads and climate, and the data outside the original input boundaries. Extrapolation, laboratory and field testing, and field performance verification may be required to use these procedures with improved reliability and confidence.

An example of empirical methods is the surface rebound deflection (Benkelman Beam Rebound or BBR) based method that was historically used by Manitoba and many other jurisdictions for pavement overlay designs. Some agencies used the BBR deflection-based method in the design for new construction projects with a staged construction technique. The staged construction involves the placement of pre-selected subbase/base layer(s) and a relatively thin AC layer in the first stage. The required additional (i.e., overlay) thickness of AC layer is then determined through deflection testing on the first stage pavement and this additional AC is placed in the second stage, typically one or two years after the construction of the first stage pavement. However, although the BBR design approach and the required data collection are quick and straightforward, the BBR data have shown very poor repeatability. Moreover, the BBR data were collected in spring which do not represent a year-round condition. The year-to-year variation of spring condition is also a major issue with the BBR data. These limitations have caused over design in some pavement rehabilitation projects in Manitoba. Therefore, Manitoba has discontinued the collection of BBR data in 2008 and the use of BBR deflection-based overlay design approach in 2016.

The most commonly used and well accepted empirical design method for both new construction and pavement rehabilitation is the AASHTO 1993 Guide for Design of Pavement Structures (ASHTO 1993) because of the experience, comfort and confidence gained by agencies over the last several decades. This method was initially developed based on road tests conducted in the 1950's and has gone through several updates since the initial development. Manitoba has been using this approach for new construction over the last several decades with some local modifications to inputs. Manitoba has revised the design inputs for new construction and has also started to use it for pavement rehabilitation design in 2017. This method is now the principal approach of pavement design in Manitoba and the primary focus of this manual.

2.4.3 Mechanistic Design Method

The mechanistic design method is based on the theories of mechanics that relate pavement structural response (deflection, strain, stress, etc.) and the accumulated damage due to repeated traffic loads. The structural response of a pavement layer depends on the fundamental properties of that layer material. Accordingly, one of the key elements of the mechanistic pavement design approach is the accurate prediction of the response of the pavement layer materials to the applied load. The linear elastic solutions provided by Boussinesq, Burmister, and Westergaard were important initial steps for the theoretical description of pavement response under an applied load (Christopher et al. 2006).

In the mechanistic design methods, stresses, strains and deflections are determined at critical locations in a pavement structure for various loads through theoretical analysis in a multi-layer system. It involves an iterative analysis of a series of trial pavement sections to identify a suitable combination of layer thickness for which the predicted critical stresses and strains do not exceed the limiting values (critical failure criteria) for each layer. This analysis theoretically ensures that the selected layer combination will be adequate to achieve the desired service life in terms of axle load repetitions (TAC 2013).

In these methods, the values of design inputs for each layer material such as elastic or resilient modulus and Poisson's ratio can be determined through in-situ and laboratory tests. Although the values of performance parameters have been developed from backcalculation of full-scale or experimental pavement sections, correction factors need to be developed and applied to allow for variation in in-service pavement response and performance (TAC 2013). However, in actual practice, highly variable correction factors are required to be applied to match the calculated responses with the measured responses or performances of different pavement structures with varying layer thickness and/or subgrade combinations, traffic loads and climatic exposures. In addition, a minor change in an input (e.g., seed modulus of a layer material or subgrade) can provide a drastically different outcome(s), which is difficult to justify and use in day-to-day pavement design and assessment practices.

The mechanistic approaches assume a linear elastic behaviour of pavement materials. The assumption of linear elastic material behaviour in the mechanistic analysis means these theoretical models are unable to predict the nonlinear and inelastic responses in terms of observed cracking, permanent deformation and other distresses that are of primary interest to the practitioners or highway agencies. A far more sophisticated material models and analytical tools are required for these analyses (Christopher et al. 2006).

Although, Manitoba has performed some mechanistic (e.g., finite element) analysis for several research projects to determine layer moduli, Poisson's ratios, stresses and strains, and then the corresponding axle load repetitions to failures, Manitoba has not used any of those approaches in any structural design of pavement because of the above specified known issues.

2.4.4 Mechanistic-Empirical Design Method

A mechanistic-empirical pavement design method combines the mechanistic and empirical approaches into a single method. The mechanistic component includes theoretical analysis and determination of pavement responses in terms of stresses, strains and deflections under a given traffic loads and environmental conditions. These responses are then empirically correlated to the pavement performance in terms of observed distresses in the field such as cracking, rutting and faulting. For example, a linear-elastic mechanical model can be used to calculate the response in terms of tensile strain at the bottom of the AC layer due to an applied axle load. The calculated strain is then empirically correlated to bottom-up fatigue cracking accumulated due to load repetitions over the desired service or analysis life of a pavement structure (Christopher et al. 2006).

Several mechanistic-empirical design approaches have been developed over last several decades. The most known approaches include the Asphalt Institute procedure for flexible pavements, the Portland Cement Association (PCA) procedure for rigid pavements, the AASHTO 1998 Supplemental Guide for rigid pavements and the NCHRP 1-26 procedures for both flexible and rigid pavements (FHWA 1993). However, the most comprehensive mechanistic-empirical design approach has been developed by AASHTO under NCHRP Project 1-37A (NCHRP 2004). This design approach is known as the Mechanistic-Empirical

Pavement Design Guide (MEPDG) and its associated software is known as the AASHTOWare PMED software (AASHTO 2020). Manitoba has been evaluating and using this approach in parallel to the AASHTO 1993 empirical approach since 2007. However, several limitations, associated with this software, have been identified in different jurisdictions and research studies including Manitoba. As such, it is not yet a widely accepted pavement design and analysis approach, especially in Canada. Manitoba, together with other interested Canadian jurisdictions and consultants, is monitoring and assessing all developments associated with the PMED software. Manitoba has also established several project sites for the calibration/validation of the distress prediction models for possible full implementation in the near future.

2.5 Pavement Distresses

All pavements experience distresses in various forms during their life cycles. Proper design, material selection and construction as well as the application of timely and appropriate maintenance and preservation treatments are keys to durable pavements with an acceptable level of ride quality and safety. Pavement distresses that are related to materials, construction, traffic loads, structural inadequacy and aging are briefly discussed in this Section.

2.5.1 Distresses in Flexible, Semi-flexible and Composite Pavements

Permanent Deformation (Rutting)

Rutting is one of the primary pavement structure and material related distresses in flexible (AC), semi-flexible (AST) and composite (AC over PCC) pavements on Manitoba highways. There are two forms of rutting: surface rutting and structural rutting. Surface rutting is limited to the AC layer, which is manifested on the pavement surface in the form of multiple waves or corrugations as shown in Figure 2.0.3. Surface rutting is the typical rutting issue for flexible and composite pavements in Manitoba.

Surface rutting usually occurs in the presence of an unstable AC mixture as it undergoes plastic flow under wheel loads. The primary reasons for instability are the AC mixtures containing fine gradation of aggregate, poor quality aggregate particles, soft asphalt binder, excessive asphalt binder and high air voids content, and the construction of a thin (inadequate) AC layer over a strong support from underlying layer(s) of pavement structure and subgrade. Manitoba AC mixes (e.g., Bit B and Bit C) that have been historically in use are considered to be finely graded mixes with low stiffness. Inappropriate asphalt binder grade was shown to be another issue for highways with high traffic loads. Manitoba's move towards the adoption of Superpave AC mixes and project specific Superpave Performance Grade (PG) asphalt binder is expected to reduce the surface rutting as well as other forms of plastic flow such as shoving.

Figure 2.0.3: Surface Rutting on PTH 1 at 1st Street, Brandon, Manitoba

Unlike surface rutting, structural rutting typically occurs in pavements with well-designed AC mixtures. It is usually distributed to each layer of a pavement structure and subgrade. Structural rutting occurs due to inadequate layer thicknesses and/or exposure to traffic loads that exceed the design traffic loads. The contribution of surface layer to the total rutting is usually small in such cases. The major portion of rutting occurs in granular layers and subgrade due to accumulated permanent deformation or shear failures under heavy and/or repeated loads. In semi-flexible (AST) pavements, this kind of rutting distress may occur, together with surface break up, due to insufficient granular layer thickness and weak subgrade condition and/or due to pavement moisture/drainage issues, especially during the spring thawing season. This kind of rutting is usually manifested as a single depression line in longitudinal direction, together with extensive fatigue or block cracking (break ups), under each wheel path. Proper characterization of subgrade and accurate estimate of traffic loads for pavement design are keys to minimizing structural rutting issue.

Bottom-Up Fatigue Cracking

A bottom-up fatigue cracking initiates at the bottom of an AC layer when the tensile stress induced by the applied load exceeds the tensile strength of the AC layer. Under repetitive loads, it is manifested as a series of interconnected (multiple short, longitudinal and/or transverse cracks) in the wheel path during the early stages of their development. Ultimately, it is transformed into a nest of cracks (multi-sided small polygons of 0.3 m or less in length on each side) resembling the skin of an alligator. Accordingly, it is also known as the alligator cracking. An example of alligator cracking on a Manitoba highway is shown in Figure 2.0.4. The possible reasons for bottom-up or alligator cracking are: a thin layer of very stiff AC mix, high air voids (low density), asphalt binder aging, low asphalt binder content, excessive load repetitions, heavy axle loads, inadequate pavement structure and moisture related stripping of AC mixes.

Figure 2.0.4: Bottom- Up Fatigue Cracking in Wheel Path on PTH 2 at Oak Bluff, Manitoba

Alligator cracking can be seen in an AC surface on some Manitoba highways; however, it is not very extensive or predominant distress until an AC surfaced pavement passes the mid-stage (30-35 years) of its 50-year life cycle. The possible reasons for low amount of bottom-up fatigue cracking on Manitoba highways is the historical use of softer (less stiff) AC mixtures than the typical asphalt mixes used in other jurisdictions. The binder rich Bituminous B and C mixtures, with fine blend/gradation of aggregates and soft grade asphalt binders, can substantially bend under the applied loads without initiating crack (due to tensile stresses) at the bottom of the AC layers. Unless exhibited soon after the initial construction due to deficiencies in the quality of AC mixes and/or construction practices, the bottom-up fatigue cracks are usually manifested once Manitoba's AC mixtures have heavily aged or passed the mid-stage of their life cycles. However, fatigue cracking is a common phenomenon for semi-flexible (AST) pavements in Manitoba due to a thin mat on the surface with a low overall structural capacity of these pavement structures.

Longitudinal (Top-Down Fatigue) Cracking

Manitoba's flexible and composite pavements seem to experience more longitudinal cracking during the early age and mid-stage than the bottom-up fatigue cracking. The exact mechanism of top-down fatigue cracking, which is also known as the wheel path longitudinal cracking, is not yet fully understood. They are manifested as long single cracks at the edges of each wheel path as well as at the centre i.e., between two rut channels (which are formed by dual tires axle units) of each wheel path. The cracks tend to run parallel to the truck wheel movement. The possible reasons are: high longitudinal (in the travel direction of truck wheels) and transverse tensile stresses at pavement surface on both edges of truck wheels, aging (hardening) of asphalt binder at pavement surface and near surface resulting in low tensile strength, a soft AC mixture that allows for the development of longitudinal surface bump due to the flow of mixes under the wheel loads, very low asphalt binder contents (inability of AC mixes to withstand tensile forces), low density (low tensile strength) of the AC surface mat, and very thick AC layer (stresses are concentrated to the surface or near-surface only) and/or very stiff base/foundation (stresses are concentrated to the surface or near-surface only). Depending on the lane width and truck wheel wander, the longitudinal cracking can also be developed at the mid lane of flexible and composite pavements. Figure 2.0.5 shows an example of longitudinal cracking on a Manitoba highway section.

It should be noted here that longitudinal cracking can also be formed due to lateral movement of unstable pavement layer and embankment materials, contraction of highway embankment material, differential settlement due to lateral variation of subgrade support, pavement structure (layer materials and their thicknesses) and/or compaction (density), flow of moisture through pavement structure and embankment/subgrade, variation of side slope between two sides of a highway, steep side slope, high superelevation, improper benching (during grade widening) and other factors. Such longitudinal cracking should not be considered as top-down fatigue cracking. These issues should be addressed through appropriate geotechnical, embankment, drainage and geometric design as well as construction practices.

Figure 2.0.5: Top-Down Fatigue Cracking on PTH 67 at PTH 8, Manitoba

Transverse (Thermal) Cracking

Thermal cracking occurs due to the shrinkage of the AC mat at low temperature or due to cyclic changes in temperature (thermal cycling) where the induced thermal stresses exceed the tensile strength of the AC mat. They are manifested as top-down cracking that run perpendicular to the centreline of pavement and spaced at a regular interval. The primary reasons for thermal cracking are: an inappropriate new asphalt binder grade to withstand the thermal stresses and an age hardened asphalt binder that can not withstand the thermal stresses. The other contributing or accelerating factors to thermal cracking are: low asphalt binder content, low AC mat density, thin AC mat, asphalt binder stripping and moisture related damage to pavement. The spacing of cracks varies depending on the AC mat quality such as asphalt binder grade, content and age, mat density, mat thickness, inter-aggregates adhesion and the friction of asphalt mat with an underlying layer.

Transverse cracking has been a predominant non-load related pavement distress in Manitoba due to high day to night and seasonal temperature differential, use of non-performance grade asphalt binder and aging of an AC layer. With the use of Superpave Performance Grade (PG) asphalt binder, the quantity and severity of thermal cracking on Manitoba highways are expected to go down in the future. Figure 2.0.6 shows an example of thermal cracking on a Manitoba provincial highway section.

Figure 2.0.6: Transverse Cracking on PTH 23 at Carmen, Manitoba

Reflective Cracking

Reflective cracking is another common type of distress in the AC overlay of flexible pavements and in the composite pavements in Manitoba, which were built initially as composite or PCC pavements overlaid with an AC layer at a later time due to faulting and roughness issues in the existing PCC surfaces (see an example in Figure 2.0.7). It occurs in the AC overlay at cracks or joint locations of the underlying AC or PCC layers due to the horizontal and vertical movements of these underlying layers. A total elimination of reflective cracking in AC overlay is next to impossible. The most effective approaches that were experienced in Manitoba to minimize reflective cracking are pulverization of existing AC and rubblization of existing PCC layers prior to the construction of new AC overlays.

Figure 2.0.7: Reflective Cracking in Composite Pavement on PTH 1 at Whitehorse, Manitoba

Several methods that have been used or are still in use elsewhere to reduce reflective cracking in AC overlays include: (a) thick (≥150 mm) mat of an AC overlay, (b) enhanced flexibility of AC overlay through the use of softer asphalt binder or additives into the new AC mixture, (c) some treatments to the existing pavement, such seal coats, crack filling, slab stabilization, prior to overlaying with a new AC layer; and (d) stress-relieving interlayers such as asphalt-rubber membranes, fabrics, low-viscosity AC mix and open-graded AC mixes (NCHRP 1982). Saw and seal in the case of composite pavement or AC overlay of PCC pavement can keep reflective cracks in uniform shape and confine at PCC joint locations.

Miscellaneous Distresses

Several other common distresses can be observed in in-service AC surfaced pavements. These include: (a) potholes due to localized deficiency in AC quality (low density, high air voids, asphalt stripping, frost heave, segregation, cracks, etc.), (b) centreline joint cracking due to poor bonding between adjacent lanes, low density at the joint area and moisture infiltration, (c) block cracking due to shrinkage of AC mat (inappropriate asphalt binder or binder aging), (d) flushing and bleeding due to excessive asphalt binder and/or over compaction of AC mat, (e) ravelling or pick outs which are manifested as dislodge of aggregate particle due stripping of aggregates from the AC mix matrix (poor aggregate-binder adhesion), stripping of asphalt binder from the

AC surface, AC mix segregation, poor compaction, etc. (f) pavement edge cracking due to low density of AC at pavement edges, weak edge support, poor drainage, heavy loads at pavement edges, etc., and (g) lane to shoulder drop off which is manifested as settlement of shoulders due to difference in pavement materials, layer thickness and construction. AST and thin AC surfaced pavements also experience break ups during the spring thawing period.

2.5.2 Distresses in Rigid Pavements

Transverse Joint Faulting

Joint faulting occurs in jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavements (JRCP) because of loss of support due to erosion or pumping out of base/subbase or subgrade material underneath the joints, repetitive vertical movement of the slabs under heavy traffic loads, poor load transfer efficiency across joints, excess or free moisture underneath joints and upward curling of PCC slabs (MEPDG 2020). It is manifested as a differential elevation between adjacent slabs at transverse joints (see Figure 2.0.8 as an example, which is produced for demonstration). Faulting can also occur at cracks due to loss of support, free moisture and loss of aggregate interlock. Variation in subgrade (foundation) strength, differential settlement of subgrade (e.g., due to longitudinal variation of embankment height or slope), variation of subgrade material type, properties and moisture condition, presence of soft, swelling and frost susceptible subgrade soils, and the variation of base/subbase material types, thickness and strength from joint to joint can cause differential faulting from joint to joint. Granular layer(s) of sufficient strength and thickness can be placed over the subgrade foundation (below the PCC layer) to reduce the differential faulting.

Figure 2.0.8: Faulting in PCC Pavement (produced using Photos from PTH 9, Manitoba)

Transverse Fatigue Cracking

Transverse cracking is not extensive in PCC pavements on Manitoba highways. Transverse cracking (see an example in Figure 2.0.9) in JPCP that are related to traffic loads and in-service environmental exposures are two types: bottom-up transverse cracking and top-down transverse cracking. A JPCP experiences bottom-up transverse cracking when a critical bending stress occurs at the bottom of a PCC slab due to a wheel load, which is placed near the edge of a PCC slab and midway between two transverse joints. It is accelerated by a high positive temperature gradient i.e.; the top surface of the PCC slab is warmer than the bottom. Under repeated loads (accumulated fatigue damage), such a crack propagates from the slab bottom to the surface of the pavement and is then manifested as transverse crack (MEPDG 2020).

Figure 2.0.9: Transverse Cracking in PCC Pavement on PTH 59S near PTH 100, Manitoba

Alternatively, the top-down transverse fatigue cracking occurs due to repeated loads from heavy truck tractors with certain inter-axle spacing (between steer and drive axles) combination, and short inter-axle spacing between trailer axles when a JPCP experiences high negative temperature gradients i.e., PCC top surface is cooler than the bottom surface. It is manifested as a transverse or diagonal crack that initiates as JPCP surface starting at the critical edge (MEPDG 2020). Transverse cracking may also happen due to inactive transverse contraction joints due to late saw cut at planned joints.

Miscellaneous Distresses

The other predominant distresses in in-service rigid pavements in Manitoba include: (a) longitudinal (see Figure 2.0.10 for an example) and diagonal cracking due to locked dowels (e.g., due to misaligned dowels, no or inadequate bond breaker and damaged or corroded dowels), inactive longitudinal joint(s) due to late saw cut, thermal expansion and contraction, weak base/subgrade support, curling and warping, etc., (b) longitudinal joint separation due to the absence or damage (because of corrosion) of tie bars, (c) freeze-thaw and moisture related damage underneath the joints, (d) joint blow outs due to joint lock up, slab expansion during hot weather and confinement of slabs as joints are filled with incompressible (sand/dust) materials, (e) lane-shoulder drop offs due to differences in pavement materials, layer thickness and construction, (f) lane to shoulder separation due to expansion and contraction, subgrade movement, moisture infiltration, etc. (g) corner breaks due to high stresses or loss of support at corners, (h) PCC spalling due to high stresses at joints and cracks (because of joints filled with incompressible materials or high stresses from heavy loads), freeze-thaw damage, segregation, inadequate consolidation, poor mix quality, misaligned or corroded dowel, etc., (i) pop outs due to the expansion, under freezing condition, of aggregates (such as shale, ironstone, limestone) containing a high absorbed moisture, and (j) alkali silica reaction (ASR). Other distresses are polishing and surface scaling. D-cracking was an issue in Manitoba in the past, but it is minimal now due to the use of smaller sized aggregates in PCC mixes.

Figure 2.0.10: Longitudinal Cracking in PCC Pavement on PR 207 (North of PTH 15), Manitoba

2.6 Pavement Performance

Pavement performance is assessed in terms of a composite measure that combines all predominant pavement distresses into a single indicator. In the MEPDG and its associated software AASHTOWare PMED, pavement smoothness, expressed in terms of International Roughness Index (IRI), is considered as the indicator of overall pavement performance. In fact, pavement smoothness indicates the functional quality of pavement which affects the ride comfort, safety and vehicle operating costs including fuel consumption, wear and tear.

In the MEPDG program, IRI depends on (is estimated from) the variation of longitudinal profile (variation of rut depths along the wheel path), quantities of transverse, alligator, reflection and longitudinal cracks, subgrade material quality, local climate and pavement age (MEPDG 2020). Subgrade uniformity and pavement construction (initial smoothness) also play a significant role in long term pavement smoothness or performance.

Some agencies have developed or adopted different composite pavement performance indicators than IRI which are functions of primary pavement distresses and the surface smoothness. These composite indicators include: pavement condition index (PCI), pavement quality index (PQI) and pavement condition rating (PCR). In the AASHTO 1993 pavement design guide, pavement performance is assessed in terms of Present Serviceability Index (PSI).

2.7 Pavement Maintenance

Maintenance treatments are applied to all pavements as they deteriorate and show localized distresses and serviceability or safety issues. In Manitoba, routine maintenance treatments are generally reactive that are applied to pavements to address specific surface distress or issues. Flexible, semi-flexible and composite pavements maintenance treatments include: pothole repairs through filling or spray patching, rout and seal cracks, fill cracks, localized thin resurfacing (asphalt patching), localized subgrade failure repairs, localized levelling and cross fall corrections, shoulder repairs, etc. Rigid pavement maintenance treatments include: localized AC patching, PCC crack stitching, joint repairs, PCC joint resealing, partial depth repairs and full depth repairs. Gravel road maintenance treatments include: regrading with or without the addition of new gravel, dust control, localized failure repair with excavation and gravel refill, and pothole repairs with gravel.

These maintenance treatments help to maintain integrity of pavements and the safety of the riders, and to delay the rehabilitation or reconstruction activities. Some maintenance treatments such as crack filling, crack rout and sealing, and pothole repairs, if performed prior to the placement of a preservation treatment or AC overlay, can extend the life of that preservation treatment or overlay through a reduction of reflective cracking or other distresses.

2.8 Pavement Preservation

Pavement preservation treatments are pre-planned activities that are applied at an early stage of distress appearance (reactive preservation treatments) or applied prior to distress manifestation at a routine interval (proactive preservation treatments) to restore or maintain overall service condition and extend the service life of pavement beyond the design service life. These treatments are applied to all hard (bound material) surfaced pavements to extend their service life to a predetermined life cycle period (50 years for all newly constructed and reconstructed pavements in Manitoba) although the initial pavement design and construction are usually based on 20-25 years service lives. The treatment type at a point (age) of a pavement life cycle is selected based on the observed or anticipated (established through past experience) overall pavement service condition or specific surface distress at that point. For example, a seal coat is applied to address and prevent from further aggravation of cracking while a micro-surfacing is applied to address surface rutting.

Flexible, semi-flexible and composite pavements preservation treatments in Manitoba include: high performance chip seals, slurry seal (not a common practice in Manitoba, other than localized repairs), micro-surfacing, micro-milling (fine milling) and ultrathin overlay of AC, thin AC overlay, and partial AC milling and AC fill/inlay. Rigid pavement preservation treatments include: dowel bar retrofit to restore load transfer efficiency at joints, diamond grinding (normally done in conjunction with partial depth slab repairs, full depth slab repairs or replacement, joint repairs and dowel retrofits) to remove faulting, improve ride quality and enhance skid resistance, and the AC overlay. These treatments last 5 to 10 years, depending on the existing pavement condition, treatment type and traffic loads.

2.9 Pavement Rehabilitation

Pavements must be rehabilitated or reconstructed when their conditions deteriorate to a level that the maintenance and preservation treatments become ineffective, unmanageable and/or very costly due to increased quantity and frequency of the required treatments. Adequate funding may not be always available under the Maintenance Program budget to keep those deteriorated pavements in good serviceable condition. Such deteriorated conditions may create discomfort in ride and safety issues leading to frequent complaints from road users, especially on freeways, expressways and primary arterials.

A rehabilitation treatment may occur before or after the end of the life cycle of a pavement depending on its structural adequacy and overall condition. A need to increase the structural capacity sometimes triggers the rehabilitation activity even when a pavement is still in a fair or good condition.

In Manitoba, rehabilitation treatments are divided into two types: Minor Rehabilitation and Major Rehabilitation. Minor rehabilitations are applied to pavements on low volume secondary or collector roads with overdue rehabilitation or reconstruction treatments. These pavements have passed their life cycle and/or are in poor serviceable conditions (some key performance indicators such as rut, smoothness and cracking conditions are in poor states) making the maintenance and preservation treatments ineffective. These pavements could not be included in major rehabilitation or reconstruction categories due to inadequate capital budget and priority to other higher class and higher traffic highways. The design service life for these pavements is 7 to 10 years and the rehabilitation treatments are limited to AC overlay, partial mill and AC overlay, mill/relay AST and AC overlay or mill/relay AST and double chip seals, and where the allocated budget permits, pulverization and relay of existing AC mat and an AC overlay.

For major rehabilitation treatments, the target service life is 20 years (pavements are generally structurally adequate for 20-25 years), which is expected to reset initial serviceability to a value or level of new construction. However, the overall life cycle, with routine maintenance and preservation treatments, will vary depending on the existing pavement's condition, treatment types, subgrade materials and project climatic conditions. Therefore, the selection of an appropriate treatment of an existing pavement based on its condition including the structural capacity and type as well as severity of experienced distresses is critical to extend its life cycle and attain the value for the money of an investment.

In Manitoba, flexible pavement structural rehabilitation treatments include: AC overlay, partial milling of existing AC and AC overlay, and cold-in-place recycling (CIR) of existing AC (partial depth) and AC overlay. For rigid pavements, typical rehabilitation treatment is structural AC overlay.

Other available rehabilitation options for rigid pavements are: bonded PCC overlay (when the existing PCC slabs are in fair to good condition) and unbonded PCC overlay (when the existing PCC slabs are in poor condition). A bonded PCC overlay with joints that match the joints in the existing pavement is a very effective approach to eliminate the reflective cracking at old pavement's joint locations. An unbonded PCC overlay with a separating interlayer over the existing rigid pavement is also an effective method to minimize reflective cracking at old pavement's joint and crack locations (NCHRP 1982).

For composite pavements, structural rehabilitation treatments include: AC overlay or partial milling of the existing AC layer and a new AC overlay. For the semi-flexible (AST) pavements, structural rehabilitation is the placement of an AC overlay.

2.10 Pavement Reconstruction

Generally, reconstruction refers to the full depth replacement of all layers of a pavement structure together with rework (may involve partial excavation, removal and replacement) and/or re-compaction of the existing subgrade. This usually occurs at the end of the life cycle of a pavement or where a typical rehabilitation treatment becomes infeasible. However, full depth replacement is not a typical practice for flexible, rigid and composite pavements in Manitoba with the exception of some localized areas.

In Manitoba, pavement structure reconstruction projects typically include: 1) full depth replacement of an existing AC or PCC surface layer with a new AC or PCC layer, 2) full depth pulverization and relay of an existing AC layer followed by the construction of a new overlying AC layer, 3) rubblization of an existing PCC surface or milling of the existing AC layer and rubblization of the underlying PCC of a composite pavement followed by the placement of a new overlying PCC or AC layer, and 4) mill and relay of an AST surface followed by the placement of a new overlying AC layer. A new granular base layer is frequently placed prior to the placement of a new AC or PCC layer, especially over a rubblized concrete. Full depth reclamation of an existing AC layer together with a portion of the underlying base layer is another strategy that yet to be tried in Manitoba. In this design manual, these type of construction practices are referred to as the partial depth reconstruction.

Full depth reconstruction of existing pavements are generally limited to: 1) localized failed areas (e.g., settlement or washout) due to subgrade failure, high moisture and drainage problems, presence of organics/peat/swamp and flood damage, 2) AST pavements that have no or a thin base layer in place, and 3) localized areas of other types of pavement with restrictions in right-of-way (ROW), width or elevation (e.g., under bridge structures or in urban areas with curb and gutter), where the required overlay structures cannot be accommodated without full depth removal of existing pavement layers and part of the subgrade.

Reconstruction (partial or full depth) usually resets the pavement condition to a new state to achieve a full 50-year life cycle with timely application of the required maintenance and preservation treatments.

2.11 Pavement Drainage

Moisture is the number one enemy of all types of pavements as it causes tremendous damage to pavements and affects their performance. Therefore, moisture exposure and pavement drainage should be considered in the pavement design, material selection, construction and maintenance practices, regardless of the design procedures (e.g., AASHTO 1993 and PMED software) used.

The potential moisture related damage is significantly higher in weak/thin pavement structures than that in the properly designed and maintained pavements. In a cold climate like Manitoba, high moisture in pavement layers due to spring thawing is a critical issue for weak pavement structures. As a result, spring weight restrictions are placed on weak roads during the spring thawing period to reduce moisture induced pavement damage in conjunction with legal axle loads. Pavements may also be exposed to a high moisture condition during other seasons due to significant rainfalls and water infiltration, high water table and seepage, standing water adjacent to pavement layer(s) and subgrade, poor drainage characteristics of pavement layer materials, inadequate pavement cross falls and clogged ditches.

Moisture in pavement structures causes reduced subgrade and granular base/subbase stiffness and pumping out of fine aggregates from granular material layer underneath the pavement surface layer resulting in loss of support for the surface layer. The excess water, combined with traffic loads, also increases the potential for early subgrade failure. These can cause cracking, faulting, settlement/depression and other deteriorations in pavements, frost heaving and swelling. Water ponding on the pavement surface causes a safety hazard due to hydroplaning. Moisture also promotes stripping in AC layer material, joint deterioration (especially in PCC pavements) and freeze-thaw related damages.

It is important to remove or intercept the flow of water, whether it comes from precipitation, melted snow, groundwater or surface infiltration, for desirable performance of pavement structures. In rural areas, side ditches adjacent to pavement structures are usually constructed to collect excess water and divert them to a nearby natural creek, river or lake. Side ditches should be constructed to a depth that ensures that any free water will always be below the subbase level. If a base/subbase layer is exposed to excess moisture from any sources, the effect of such excess moisture condition should be accounted for in the design using an appropriate structural value for that layer material, or measures should be taken (e.g., with construction of appropriate drainage system) to promptly remove the excess water from the pavement layers. Urban drainage systems with curbs and gutter, catch basins and sub-drains allow for the drainage of rain and snow/ice melted surface water. However, water infiltration into pavement layers can still occur through pavement surface cracks and joints. As such, the overall subsurface drainage system should allow for the collection (e.g., with the use of base and subbase materials with good drainage quality, in addition to the standard drainage tubing) and prompt removal of excess water from the pavement layers.

It is also important to consider subsurface drainage in areas where springs and seepage conditions are encountered, where adverse frost conditions are present and subgrade soils are exposed to moisture during the freezing season or where the subgrade is susceptible to expansion or strength loss due to increased water content in all types of x-sections (rural, semiurban or urban). If the top part (the top one meter is the most critical area) of the subgrade/embankment is exposed to excess moisture from any sources, the effect of such excess moisture condition should be accounted for in the design using an appropriate resilient modulus of the subgrade and/or provision of adequate drainage system to promptly remove the excess water. Figures 2.0.11, 2.0.12, and 2.0.13 show typical subsurface drainage techniques that can be used, where required.

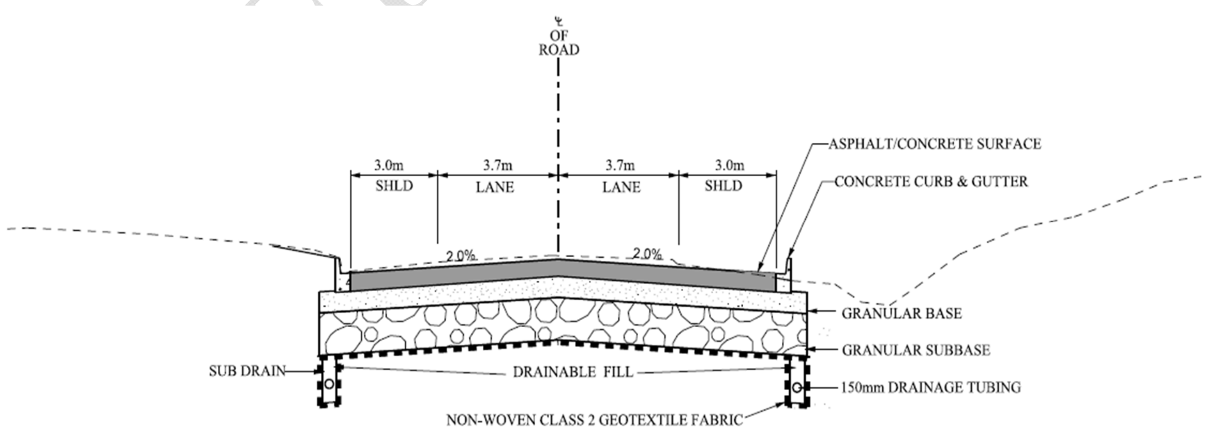

Figure 2.0.11: Typical x-Section and Subsurface French Drain Installed under Curb and Gutter of Urban Highways/Roads

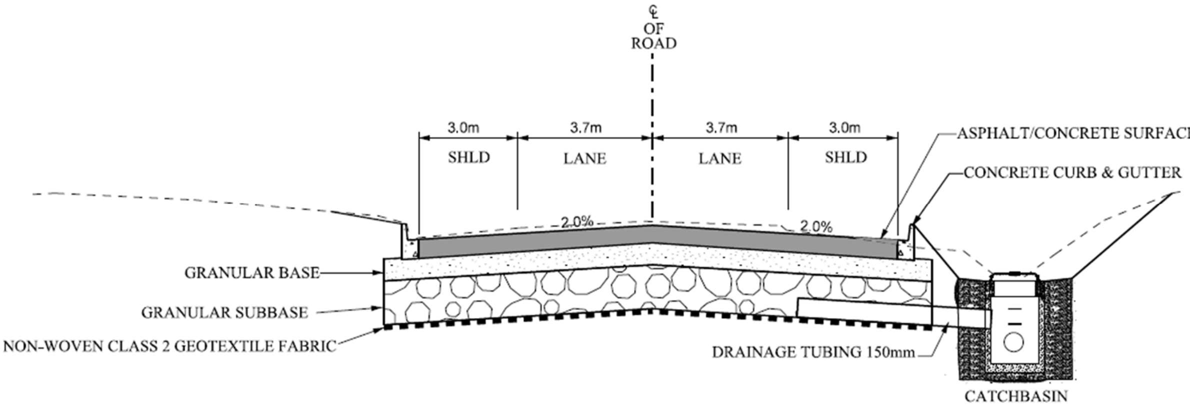

Figure 2.0.12: Typical x-Section and Subsurface French Drain Connected to Catch Basin at Urban Highways/Roads

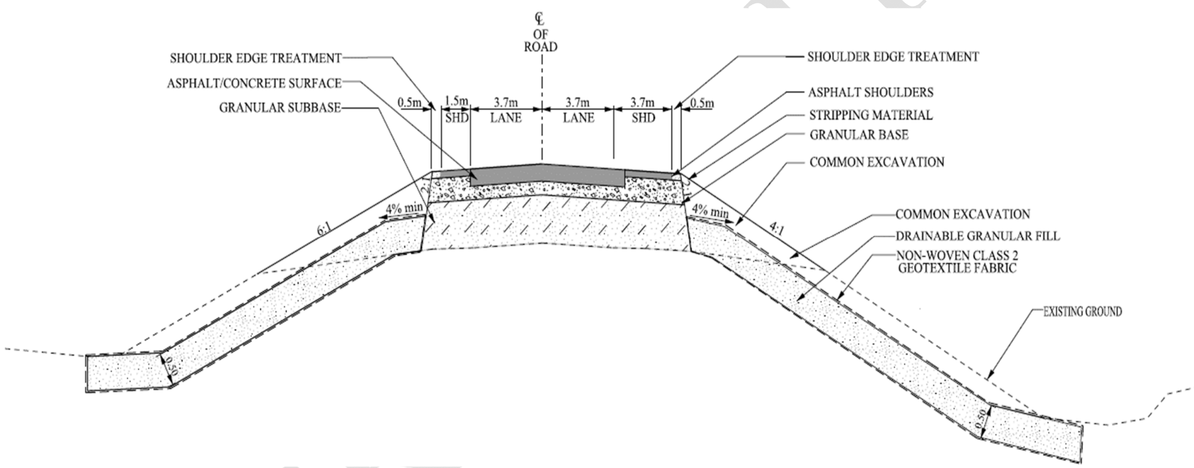

Figure 2.0.13: Typical x-Section and Subsurface Sand Drain (Embankment Drain) Installed in the Embankment of Flood Prone Highway/Road Section

2.12 Pavement Surface Type Selection

The selection of pavement surface types for highways with a high traffic volumes and/loads (e.g., freeways, expressways and primary arterials) is often based on an economic analysis of alternative pavement options. This applies mostly to new construction and reconstruction projects. For highways with low traffic volumes and/or loads, the selection of pavement surface types is typically based on the functional classification and traffic volume. However, some jurisdictions also select the most cost-effective rehabilitation treatment based on the life cycle economic analysis of alternative treatments.

Table 2.0.1 provides generic guideline for the selection of surface types on Manitoba highways based on highway functional and strategic classifications and traffic volume. An economic analysis of alternatives, where applicable, is desirable to select the most cost-effective option, unless an option is chosen based on initial construction costs due to budget limitation during the initial construction phase. In addition, respective region should be consulted for the final selection of pavement surface type, regardless of traffic volume, specially for service roads. All highways/routes with the strategic classification of Trade and Commerce should be AC or PCC surfaced and designed for RTAC loading classification regardless of traffic volume and functional classification.

Table 2.0.1: Selection of Pavement Type Based on Functional and Strategic Classifications

Highway Functional and Strategic Classifications	Projected 20-Year AADT	Pavement Type
Freeway	No limit	AC or PCC
Expressway	No limit	AC or PCC
Trade and Commerce Route	No limit	AC or PCC
Primary Arterial (other than Trade/Commerce Routes)	No limit	AC or PCC
Secondary Arterial (other than Trade/Commerce Routes)	≥ 500	AC or PCC
	< 500	AST[1] or AC[2]
Collector and Access Roads (other than Trade/Commerce Routes)	≥ 1000	AC or PCC
	< 1,000	AC[2]
	300 – 1,000	AST[1] or AC[2]
	< 300	Gravel (Granular)

Note 1: Standard surface type for new construction. Subject to weight restrictions during the spring thawing season if an AST surface is placed over a thin (less than 300 mm) base layer.

Note 2: Standard surface type for rehabilitation or reconstruction of an existing AST pavement, unless an alternative is chosen due to budget constraint. Spring weight restriction should be removed with the placement of specified minimum AC and base/subbase layers. Subject to weight restrictions during the spring thawing season if less than 85 mm AC surface is constructed over a thin (less than 300 mm) base layer, unless confirmed otherwise through post-construction surface deflection testing using a falling weight deflectometer (FWD).

An economic analysis process compares life cycle costs, and occasionally benefits, of different pavement/surface types for new construction and reconstruction or alternative treatments and the rehabilitation/resurfacing options for an existing pavement to objectively select the most suitable one for highway construction project. Such analysis must be logical and reasonable so that the selection is based on the cost effectiveness of the available alternatives. Several analysis methods are in use in different jurisdictions in North America, which are briefly discussed in the following section.

2.12.1 Life Cycle Cost Analysis (LCCA)

Highway investment, whether it is in the form of capital construction for new roads or for maintenance, preservation, rehabilitation or reconstruction of existing roads, represents an effort to make road transportation as safe and comfortable as possible to the users. Expenditures for a particular roadway, including all construction and maintenance and preservation activities, are spread over the life of the road until the road becomes candidate for another major rehabilitation or reconstruction. These expenditures are referred to as the Life Cycle Costs. To complete a life cycle cost analysis, the influence of interest, inflation and discount rates should be considered. It is also important to select an analysis period that includes at least one overlay activity and its service life. The department currently uses a design service life of 20 years for flexible and 25 years for rigid and composite pavements, and a 50-year life cycle analysis period for these pavements. A shorter life cycle period (say, 30 years) could be chosen for a comparative economic analysis of chip seal, gravel and thin AC surfacing options as well as for comparing alternative rehabilitation treatments of an existing pavement, e.g., AC overlay, mill and overlay, and cold-in-place recycle and overlay. For comparing the life cycle costs of different rehabilitation and reconstruction options, differing life cycles could be chosen for different treatments, e.g., 30 years for rehabilitation and 50 years for reconstruction options.

Historically, Manitoba has been performing the LCCA to compare between AC and PCC surfacing options on major projects. The LCCA of other alternative surfacing and existing pavement treatment options for other highway capital projects is not yet developed or used in Manitoba. The surfacing or treatment options on these projects are basically based on the initial construction costs, available time and the available budgets. Manitoba plans to develop a new LCCA strategy and procedure for all projects including the pavement rehabilitation. Details of this new strategy and procedure will be covered in a separate guideline.

When evaluating a pavement structure or surfacing option, there are five major cost components that are to be considered, which are:

Initial Construction Costs

The initial construction costs include all costs associated with a pavement structure option including equipment mobilization, excavation, embankment, shoulders and traffic control. Costs for each structure option is based on the unit price of each item that are prevalent in the area and, where applicable, extrapolated to the anticipated construction date based on the inflation rate of construction costs.

Overlay Costs

Overlay costs include the costs of future overlays or other upgrading required when the pavement condition or serviceability reaches a specified minimum level of acceptability.

Maintenance and Preservation Costs

Maintenance and preservation costs include only those items which directly affect pavement performance. Attention should also be given to the increased annual maintenance with increased pavement age.

Salvage Value

Theoretically, salvage value should include values of materials which can be salvaged and reused, and it should consider the deterioration in the quality of materials during the life of the original pavement structure and various treatments. It should also account for the added costs associated with the removal and processing of materials for reuse, and disposal of removed materials, as applicable. Alternatively, simply the remaining life of a treatment, if it will remain in place at the end of the analysis period, can be taken into consideration for determining the salvage value.

User's Costs

The road user's costs include: vehicle operating costs, travel time costs, traffic delay costs due to the construction, maintenance and preservation activities, traffic collision costs and discomfort costs. User costs are affected by ride quality, vehicle speed and safety features such as sight distance and pavement surface distresses.

2.12.2 Benefits

The benefits attached to a road network improvement program can be classified as either direct or indirect benefits.

Direct Benefits

Direct benefits that result from any improvement to the road network are: reduced travel time, reduced vehicle operating costs, improved access to land and development adjacent to the project site, improved safety and improved economic activities through increased and/or efficient movement of goods as well as services and tourist attraction.

Indirect Benefits

Indirect benefits include: the benefits attached to the salvage and reuse of materials, lower price for materials and construction due to competitive alternative options, reduced commodity and service prices due to efficient or uninterrupted movement of goods and services, higher revenue of governments from taxes and duties due to increased economic activities, improved socio-economic factors for affected communities and greater life expectancy.

2.12.3 Economic Evaluation Methods

To compare different pavement construction and rehabilitation alternatives, it is necessary to identify the difference in the worth of money spent over the life cycle of each alternative. In addition to the current costs of construction and the unit prices of materials and treatments (may include benefits and road user's costs), the expected future interest rate on borrowed money, accrued interest rate on short term bank deposit, inflation rate of unit prices of materials and treatment activities, and the calculated discount rate are key inputs to the life cycle economic analysis. Several models are available to carry out this economic analysis. Three models that are commonly used are:

Present Worth Method

This method attempts to compare overall costs of alternative options at present value of the money spent over the life cycle of each option. It involves the discounting of all future costs to the present value using an appropriate discount rate. The life cycle period of competing options should be the same to use this method.

Equivalent Uniform Annual Cost Method

This method combines all initial capital costs and all recurring future expenses into equal annual payments over the analysis period using an appropriate interest rate on borrowed money for infrastructure. The life cycle period of competing options could be different in this method. This method can also capture the interests cost on borrowed money.

Benefit-Cost Ratio Method

The benefit-cost ratio method compares the ratio of the present worth of all benefits to the present worth of all costs, or the ratio of the equivalent uniform annual benefits to the equivalent uniform annual costs of an alternative to other feasible alternatives.

2.12.4 Issues and Alternatives

While it is relatively easy to determine direct costs to the department associated with the initial construction and life cycle maintenance as well as preservation treatments, determining the benefits and road user's costs are not that easy. As such, Manitoba to date has not included benefits and road user's costs in the analysis. Although some models are available to quantify the user's costs and benefits, no generally accepted model is available yet across North America. The department plans to include user costs in LCCA in the future when generally accepted input data and model(s) are available.

An alternative to the benefit-cost analysis is the determination of cost effectiveness, which simply accounts for the life cycle costs of each alternative and its service benefits in terms of traffic uses with a desirable level of service. It will indirectly account for all the costs and benefits to the department as well as the users. Details of this procedure will be covered in the planned new LCCA guideline.

Although the life cycle economic analysis will provide a basis for decision-making, several additional factors need to be considered together with life cycle costs for rational decision-making. These factors include, but are not limited to, road geometrics, materials availability, budgets, maintenance levels, interruptions to travelers, route as well surface type continuity, public perception, drainage, safety, climate, past experience with similar pavements and good engineering judgment.

2.13 Sustainability and Climate Change Consideration

According to the World Commission on Environment and Development (WCED), sustainable development refers to "*the development that ensures that it meets the needs of the present without compromising the ability of future generations to meet their own needs*" (WCED Report 1987). In turn, it refers to both of sustainable uses of resources and adoption of practices for development that preserve or minimize depletion of natural resources and minimize negative impact on natural environment and climate. However, with centuries of uncontrolled or poorly managed development and use of resources, we have abused our natural system and resources leading to climate change and fast depletion of non-renewable resources. We are late to act but should not sit idle, which can make things worse.

When it comes to highway construction, sustainable development and construction practices should include the efficient (i.e., reduced) use of resources (such as natural/virgin materials, equipment/tools/vehicles and fuel/energy), waste minimization (i.e., reuse, recover and recycle, etc.) and substitution (i.e., the use of an environmentally sound and an economically viable substitute). Pavement design and construction should consider zero waste of materials including 100% use of the reclaimed materials from roadways such as existing or reclaimed asphalt pavement (RAP), existing or reclaimed PCC, salvaged granular base/subbase layer materials, and where applicable, the salvaged embankment materials. The reclaimed materials should undergo appropriate processing/recycling i.e., properly engineered to meet requirements that are applicable to an engineering material. The engineering requirements of a material include stiffness, long term stability, long term durability, workability during the placement and future recycling potential. Some potential uses of existing roadway and reclaimed materials are presented in Chapter 3 (Pavement Materials).

In addition to the sustainability consideration, pavement design, construction and maintenance practices should consider the impact of climate changes on pavement performance and incorporate climate resilience into those practices. The FHWA Tech Brief (FHWA 2015) outlines several climate change adaptation measures for surfaced pavements. The proposed measures consider the impacts of changes in key climatic parameters such as temperature, precipitation, and sea level. The temperature parameter includes higher average temperatures, higher extreme maximum temperature, warmer extreme minimum temperature, reduced freezing days and potential more freeze-thaw events. The precipitation parameter includes more extreme rainfall events, higher average annual precipitation, wetter winters and drier summers, and lower summer humidity. The impact of sea level is the rise in sea level that will mainly affect the infrastructures in coastal areas.

The FHWA Tech Brief (FHWA 2015) also provided a list of key pavement performance indicators that should be monitored for climate change impacts. For flexible pavements, the relevant indicators are: rutting on pavement surface, low temperature (thermal) cracking, block cracking, raveling, fatigue cracking, potholes, rutting of subgrade and unbound granular material, and stripping in asphalt layer. For JPCP, the relevant indicators are: blow-ups, slab cracking, joint spalling, freeze-thaw durability, faulting, pumping and corner breaks, and slab warping. Mayer et al. (2014) recommended some practices that can be taken into consideration when designing AC and PCC surfaced pavements to adapt with the potential changes in above specified climatic parameters. To make pavement structures resilient to potential impact of climate changes in Manitoba, those recommendations have been tailored, with some modifications, additional considerations and required analysis, to suit Manitoba's local experiences and needs. The analysis and measures that are to be considered in pavement design, construction and management practices to adapt with potential impact of climate changes are listed in Tables 2.0.2 and 2.0.3. Relevant other engineering/technical areas should also be involved in the project design including hydrological analysis, hydraulic analysis, drainage design, culvert design and roadside as well as ROW treatments and features.

It should be recognized here that the assessments specified in Tables 2.0.2 and 2.0.3 for considering the effect of climate changes on different pavements are beyond the day-to-day pavement design/analysis and treatment selection practices. The department will analyze/assess the trends of the specified and any other relevant climatic parameters that will affect pavement performance and establish the inputs and requirements at certain interval (e.g., every 10 years) for consideration in pavement design/analysis and treatment selection. Such recommendations will be reflected in an Engineering Standard.

Table 2.0.2: Analysis and Measures for Climate Change Adaptation- Temperature Change

Climate Change Impact and Required Assessment and Analysis	Recommended Measures (if warranted)
<u>Higher Average Temperatures</u> • Assess the trends for seven-day average maximum pavement surface and effective pavement temperatures for the last 30 years. • Predict the potential increase in seven-day average maximum pavement surface and effective pavement temperatures over the next 30 years.	**<u>AC Surfaced Pavements</u>** • Raise the high temperature grade of asphalt binder to withstand rutting and shoving, and/or improve aggregate and mix qualities to make AC mixes more resistant to rutting and shoving. • Seal the AC surface sooner to slow aging of the asphalt binder. **<u>PCC Surfaced Pavements</u>** • Consider shorter joint spacing based on coefficient of thermal expansion (CTE) of PCC to limit slab cracking due to curling. • Consider high friction base surface to increase resistance to curling. • Consider the use of microfibers and appropriate chemical admixture to avoid cracking due to drying shrinkage. • Pay more attention to PCC curing process and protection to avoid cracking due to drying shrinkage or rapid hydration.
<u>Higher Extreme Maximum Temperatures</u> • Assess the historical (last 30 years) trends of maximum (pavement surface and subsurface) temperatures and potential increase in maximum temperatures over the next 30 years. • Assess the historical (last 30 years) trend of drought conditions and project the potential increase or worsening of drought conditions for the next 30 years.	**<u>AC Surfaced Pavements</u>** Consider the above specified measures for Higher Average Temperatures plus the following: • Stabilization (cement, lime, etc.) of clayey subgrade to limit potential shrinkage of subgrade due to drought. **<u>PCC Surfaced Pavements</u>** Consider the above specified measures for Higher Average Temperatures plus the following: • Stabilization (cement, lime, etc.) of clayey subgrade to limit potential shrinkage of subgrade due to drought. • Shorter joint spacing to eliminate potential blowouts. • Routine cleaning of joints to limit potential blowouts. • Add expansion joints to limit potential blowouts. • Consider paving at night.

<u>Extreme Minimum Temperature</u> • Assess the trends for daily minimum pavement surface temperature and frost penetration into pavement structures in winter for the last 30 years. • Predict the potential increase in the minimum pavement surface temperature and decrease in frost penetration depth over the next 30 years.	**<u>All Pavements</u>** • Reduced frost protection may be considered if reduction in frost penetration depth shows a consistent trend with due consideration of occurrences of abnormal extreme low temperature events and the resulting increased frost penetration. **<u>AC Surfaced Pavements</u>** • Warmer minimum pavement temperature may warrant warmer low temperature grade of asphalt binder, but it should not be considered at this time to avoid cracking due to occurrences of abnormal extreme low temperature events.
<u>Increased Freeze-Thaw and Mid-winter Thawing Events</u> • Assess the trends for freeze-thaw and mid-winter thawing events and the frequency of de-icing salt application for the last 30 years. • Predict the potential increase in freeze-thaw and mid-winter thawing events, and the frequency of de-icing salt application over the next 30 years.	**<u>AC Surfaced Pavements</u>** • Specify asphalt binder requirements to ensure that it can withstand increased thermal cycles. • Adjust the structural layer coefficients of base and subbase materials, as applicable, with the application of appropriate monthly factors for resilient moduli values to account for the thaw weakening in winter and increased thaw weakening in early spring. **<u>PCC Surfaced Pavements</u>** • Adjust PCC mix design to ensure that PCC can withstand increased freeze-thaw cycles and de-icing salt application.

Table 2.0.3: Analysis and Measures for Climate Change Adaptation- Precipitation Patterns

Climate Change Impact and Required Assessment and Analysis	Recommended Measures (if warranted)
<u>Increased Extreme Rainfall Events</u> • Assess the historical frequency of extreme rainfall events (last 30 years) and project its future (next 30 years) frequency on monthly basis. • Assess the adequacy of pavement surface skid resistance. • Assess the pavement cross slope for adequacy of surface drainage and potential for hydroplaning. • Assess risk of embankment failures (washout, reduced structural capacity, etc.). • Assess ditch and culvert capacity. • Assess the visibility of pavement/lane markings.	**<u>All Pavements</u>** • Design pavement surface layer or apply appropriate preservation treatment to provide and maintain improved surface texture and skid resistance. • Provide and maintain (with the required maintenance, preservation or rehabilitation measures) adequate cross slope to facilitate quick flow of water from pavement surface and reduce the risk of flooding, hydroplaning, splashing/spray and road/embankment slope failure. • Increase rutting resistance of pavement with stiffer surface layer and/or reduce the threshold value of required rut depth for applying the rut fill preservation treatment (not applicable to rigid pavements). • Apply adequate tack coat between lifts/layers of asphalt/emulsion bound materials to eliminate potential risk of delamination. • Increase ditch and culvert capacity to reduce water pressure on embankments around culverts and potential for washout of embankments. • Consider increasing the pavement surface elevation to prevent flooding, where ditch and culvert capacity can not be increased. • Provide and maintain functioning sub-drainage to provide quick drainage of water from pavement structures. • Consider stabilized subgrade and subbase materials to improve stability of embankments and prevent washouts. • Reduce the layer moduli of unbound base, subbase and subgrade when they are submerged or saturated and factor them in calculating the effective resilient moduli and structural layer coefficients. • Consider flattening the side slopes (e.g., 6:1 instead of typical 4:1) to reduce potential for embankment erosion. • Consider alternate pavement marking to improve visibility of lane demarcation.

Higher Average Annual Precipitation • Assess the historical (last 30 years) and project the future (next 30 years) average annual and monthly precipitation. • Assess the moisture condition, moisture susceptibility and drainage quality as well as condition of granular base, subbase and subgrade materials. • Assess the anti-stripping quality of surfacing and bound (e.g., emulsion, asphalt, cement treated stabilized) materials. • Assess the pavement cross slope for adequacy of surface drainage. • Assess ditch and culvert capacity. • Assess the adequacy of pavement surface elevation to prevent flooding of surface, especially during the snow melting season.	**All Pavements** • Revise the effective layer moduli of unbound base/subbase layers and subgrade based on the adjusted monthly factors for moduli variation, which should be determined through FWD deflection testing. • Consider stabilization of subgrade and subbase materials or the use of granular material for subgrade/embankment construction to reduce moisture susceptibility without compromising resistance to erosion. • Provide adequate subsurface drainage for unbound materials and subgrade to reduce moisture exposure and susceptibility to weakening. • Increase ditch and culvert capacity to reduce water pressure on embankments around culverts and washout of embankments. • Consider flattening the side slopes (e.g., 6:1 instead of typical 4:1) to reduce potential for embankment erosion. • Consider increasing the pavement surface elevation to prevent flooding. • Consider antistripping agent(s) in all highways to reduce/eliminate potential stripping of asphalt mixes. • Apply adequate tack coat between lifts/layers of asphalt/emulsion bound materials to eliminate potential risk of delamination. • Provide and maintain (with the required maintenance, preservation or rehabilitation measures) adequate cross slope to facilitate quick flow of water from pavement surface. • Consider materials and construction processes that are less susceptible to weather-related delays.
Wetter Winters and Drier Summers • Assess the historical (last 30 years) and project the future (next 30 years) drought conditions and drought durations in late spring, summer and fall. • Assess the historical (last 30 years) and project the future (next 30 years) number of wet days, freeze-thaw events and potential for moisture changes in granular base and subbase in winter and early spring.	**All Pavements** • Address the increased potential for soil shrinkage and swelling due to moisture changes, particularly in time of drought, with embankment design and material selection. • Consider soil stabilization to reduce or eliminate shrinkage and swelling potential. • Consider reduced effective stiffness of (or support from) base and subbase materials with the application of appropriate monthly factors for resilient moduli values to account for the thaw weakening in mid-winter and increased thaw weakening in early spring.

• Assess the potential for increased shrinkage of soils and swelling due to moisture changes. • Asses the freeze-thaw resistance of PCC materials. • Assess the potential increased risk of PCC saturation during the critical freeze-thaw cycles and increased de-icer use.	• Quickly clear the wet snow from roadway surface and remove snow/ice from roadway shoulders to allow drainage of melted wet snow from roadways and shoulders. **<u>Flexible Pavement</u>** • Provide stronger pavement structures that are less susceptible to changes in subgrade properties due to changes in moisture condition. • Specify asphalt binder requirements to ensure that it can withstand increased thermal cycles. **<u>Rigid Pavement</u>** • Design PCC joints (e.g., use non-corrosive dowels and tie bars and seal/fill joints as well as cracks) and provide drainage to ensure that the PCC remains below critical saturation. • Improve the resistance of PCC to freeze-thaw and de-icing salts with the use of appropriate materials and mix designs.
<u>Low Summer Humidity</u> • Assess the historical (last 30 years) and project the future (next 30 years) humidity conditions (levels and durations) in late spring, summer and fall months. • Assess the potential for increased aging of asphalt binder (due to increased volatilization) because of low summer humidity together with hotter summer temperatures. • Assess the potential increase in PCC slab warping over a long-term.	**<u>Flexible Pavement</u>** • More frequent application of preservation treatments to reduce asphalt binder aging. • Modify asphalt mix design and binder type/grade selection to address potential for binder aging. • Use pavement preservation materials/techniques that reduce asphalt binder aging. • Use asphalt binders with additives that age more slowly. **<u>Rigid Pavement</u>** • Enforce good curing practices during PCC pavement construction to avoid risk of drying shrinkage and ensure proper hydration. • Reduce drying shrinkage of PCC mixes by decreasing paste volume without sacrificing the strength and properties that are required to withstand traffic loads and climatic exposures or by using appropriate chemical admixtures. • Reduce PCC slab length, if needed, to reduce the severity and frequency of cracking due to PCC drying shrinkage.

Chapter 3: PAVEMENT MATERIALS

3.1 Overview

Pavements are generally layered structures which are constructed over built embankments or prepared subgrade consisting of native and/or borrowed soils. Accordingly, pavement design and construction usually involve several types of materials. The pavement construction materials can be grouped into four major categories: surface, base and subbase courses, and the supporting subgrade materials. The materials used for pavement maintenance and preservation treatments are usually different from those used for new construction, reconstruction and rehabilitation. Pavement surface layers, except for gravel roads, usually consist of bound materials such as AC, portland cement concrete (PCC) and chip seal. The base, subbase and subgrade materials could be unbound (i.e., untreated soils and aggregates) or bound (e.g., cement stabilized soils and aggregates). However, unbound materials are commonly used for base, subbase and subgrade construction in Manitoba.

The performance of a pavement structure or treatment also depends on the quality and placement of materials, in addition to the layer thickness of each material. A sound knowledge of the engineering properties of different materials and their performance or the suitability in different traffic and climatic exposures, and for different applications are critical to ensure the desired service life or life cycle of pavement structures. In addition, the induced stress in a pavement structure, due to traffic and environmental loads, decreases with increased depth from the pavement surface with the maximum stress being induced at the surface of the pavement. The material(s) at or near the surface are also exposed to traffic wear and environmental degradation. Therefore, it is important to use good quality (stiff, wear resistant and durable) materials at and near the pavement surface. An inferior quality material(s) should be placed at deeper depths (e.g., in subbase layer or fill) where the induced stress is lower than that at the surface or near the surface.

The types of pavement materials used by an agency also can vary over time due to changes in specifications based on research and past performance experience. As a result, the materials that are in place in the existing pavements could be different from those that are currently being used. The quality of in-situ materials can also degrade over time due to traffic and environmental exposures. Therefore, it is important to gain a thorough understanding of the in-situ materials before providing designs for pavement rehabilitation and partial depth reconstruction projects or for reuse of those materials.

Manitoba had been historically using several AC mixes such as Road Mix, Bituminous B (Bit. B) and Bituminous C (Bit. C). Although Bit. B mix is still in use, whereas Road Mix and Bit. C mix were discontinued long ago, Manitoba has developed new specifications in 2019 to adopt SuperPave mixes and started to switch to SuperPave mixes with an initial focus on using them for freeways, expressways and primary arterials. When it comes to the asphalt binder, Manitoba has completely switched from the historical use of viscosity and penetration grade asphalt binders to the SuperPave Performance Grade (PG) asphalt binder in 2018 (partial implementation started a decade ago). For unbound granular base and subbase, Manitoba had been using A base, C base, Modified C base, granular fill of varying gradations, and crushed rock of inconsistent specifications. The stiffness, drainage performance and stability of these granular materials had become concerns from an engineering point of view and based on field performance experience throughout Manitoba. The specifications for surface granular (traffic gravel) also varied widely. In 2019, Manitoba developed new specifications for base, subbase, crushed rock, granular fill and surface granular materials and fully implemented the new specifications in 2020. The PCC mix specification has evolved over time and it is still under further refinement for making Manitoba's JPCP more economical and durable.

The subgrade material, which is the foundation of a pavement structure, and which ultimately bears the stress from the applied traffic loads, varies widely in Manitoba. The subgrade material varies based on soil types and composition, classification, contents such as moisture, organics and peat, and behaviour such as frost susceptibility and swelling potential. Proper characterization of the subgrade material at each specific project site is critical to ensure the desired performance and service life of the pavement structure, whether it is new construction, reconstruction or rehabilitation.

3.2 Subgrade Soils

In Manitoba, subgrade soil type and composition can vary widely from location-to-location and even in a short distance interval within a project. A thorough investigation of soil types, contents and properties including their variation from point to point in each project, especially for a new grade, is critical for a reliable pavement structural design and analysis, and for highway construction in an economical manner. For an existing grade, pavement deflection data can be collected using a falling weight deflectometer (FWD) equipment and used to determine the stiffness of subgrade soils by backcalculation. However, assessment for frost and swelling potential requires soil classification and characterization into various frost heave and swelling categories and severity levels as well as for frost heave and swelling rates. The soil survey plan should be developed considering the project type (new construction, reconstruction and rehabilitation), available data (FWD and past soils survey data), local experience of frost heave/settlement, swelling, organics/peat issues and the available resources to complete the surveys and testing in a timely manner.

3.2.1 Subgrade Soil Survey

A soil survey is conducted to determine the classification of soils, layer depth and thickness of each soil type, and soil contents such as organics, peat, topsoil and moisture. Some soil surveys involve the determination of depth to the groundwater table and the depth to as well as the extent of bedrock. Soil survey is usually conducted by drilling boreholes at specified frequencies and to specified depths, but they should vary to determine the extent of major change in soil type, soil contents and the problematic soils within the project limit. A ground survey may be also required when bedrocks are exposed to the ground surface to determine bedrock topography. Samples of soils from boreholes are collected for laboratory testing. The Department's Engineering Standard ENG- PG001 "*Soil Survey for Design and Assessment of Highway Pavements and Embankments*" outlines the requirements for soil survey including sampling and testing for different applications. Potential variation of soils on existing pavement shoulders from that on the main lanes should be determined by drilling additional boreholes on shoulders in pavement rehabilitation and reconstruction projects. Additional soil sampling and testing will be required for road alignments to determine the maximum dry density, optimum moisture content and resilient modulus or soaked California Bearing Ratio (CBR) value of subgrade soils where no FWD data is available (e.g., all new grades and gravel roads).

In a soil survey program, particular attention should be given to the determination of frost susceptibility and swelling potential of soils and their impacts if such issues were encountered in the past at the project location. Site information should include the frost heave interval (average linear distance), frost severity (how bad is the issue) and frequency of occurrence (how often it occurs). The frost heave or swelling rate, as applicable, should be estimated and used in the analysis, design and recommendation. The average depth of frost penetration below the pavement surface in the project area should be obtained from the thermistor data, where available, or estimated from the historical average cumulative freezing index (CFI) data.

As part of the investigation work, the soil survey team should also identify any subsurface lateral drainage issues and include them in the report.

3.2.2 Subgrade Soil Classification

Soils are usually characterized based on engineering properties such as grain size distribution and Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index), which are determined in the laboratory. Soils are then classified into different groups and classifications in accordance with the Unified Soil Classification System (USCS) or the AASHTO soil classification system. The AASHTO soil classification system classifies subgrade soils into different groups and classes for use in highway construction purposes while the USCS is primarily used to classify soils for geotechnical purposes. However, many highway agencies also use the USCS for pavement design purposes. Manitoba has been historically using the AASHTO soil classification system and started to use the USCS approach in 2018 to assess the subgrade soils frost susceptibility.

AASHTO Soil Classification System

The AASHTO system of soil classification (AASHTO M145) is designed so that subgrade soils may be classified into groups by means of visual inspection and simple laboratory tests. The soils are first divided into two general classifications: Granular Materials and Silt-Clay Materials. Subgrade soils containing 35 percent or less fine particles (smaller than 75 µm diameter) by weight are considered granular materials. The granular materials are further subdivided into three basic classification groups (A-1, A-3 and A-2) and seven subgroups (A-1-a, A-1-b, A-3, A-2-4, A-2-5, A-2-6 and A-2-7). The silt-clay materials, which contain more than 35 percent fine particles by weight, are subdivided into four basic classification groups (A-4, A-5, A-6 and A-7). The A-7 group is further divided into two (A-7-5 and A-7-6) subgroups. Soils having the same general strength and service characteristics are grouped together to form the above stated seven basic soil groups:

1) A-1 Group: Gravel and Coarse Sand;
2) A-3 Group: Fine Sand;
3) A-2 Group: Silty or Clayey Gravel and Sand;
4) A-4 and A-5 Groups: Silty Soils; and
5) A-6 and A-7 Groups: Clayey Soils

Soils that fall in the A-1 group (A-1-a and A-1-b subgroups) are considered as the most suitable materials for highway embankments and subgrade. The increasing numerical order of soil class designation generally reflects poorer soils with soils in the A-7 group being the poorest subgrade soils, with the exception of the A-3 group. Soils that fall in the A-3 group are better subgrade soils than those in the A-2 group provided that A-3 soils are protected or confined to stop free movement. It is generally understood that these soil classification groups reflect the relative strength of the soils. However, this assumption may not always hold, especially for the silt-clay materials, due to the variation in other contents (moisture, organics, etc.), confinement and inter-particles adhesion or bonding.

It should be noted that the soils in the A-4 group are non-plastic or moderately plastic silty materials which are sensitive to moisture variation. They can be unstable materials when exposed to moisture, and highly frost susceptible when exposed to moisture as well as freezing temperature. The soils in the A-6 group are typically high plastic materials which exhibit high dry strengths but go through volume change with change in moisture content and can be compressed when wet (TAC 2013).

In Manitoba, soils that fall in the A-7 group are highly plastic and are usually soft clay material with high moisture content. These materials are also known to create frost and swelling issues.

Highly organic soils, such as peat, are not included in this classification because of their undesirable properties. Their use should be avoided, if possible, in all types of construction. The AASHTO soil classification system is presented in Table 3.0.1.

AASHTO Group Index

The AASHTO group index (GI) is typically used as a general guide to indicate the relative strength of a soil. It is a function of liquid limit, plasticity index and the percentage of particles smaller 75 µm size (passing No. 200 sieve). The percentage passing the 75 µm sieve is based only on the sample material passing the 75 mm sieve. The AASHTO group index can be calculated using the following formula (Equation 3.1):

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10) \qquad (3.1)$$

where,

F = percentage passing 75 µm (No. 200) sieve, expressed as a whole number
LL = liquid limit (%)
PI = plasticity index (%)

Table 3.0.1: AASHTO Classification of Highway Subgrade Materials (Adopted from AASHTO M 145-91)

General Classification	**Granular Materials (35% or Less Passing #200 Sieve)**							**Silt-Clay Materials (More Than 35% Passing #200 Sieve)**				
Group Classification	**A-1**		**A-3**	**A-2**				**A-4**	**A-5**	**A-6**	**A-7**	
	A-1-a	**A-1-b**		**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**				**A-7-5**	**A-7-6**
Gradation, % Passing												
Sieve Size (mm) / **Sieve #**												
2.0 / **10**	50 max.	-	-	-	-	-	-	-	-	-	-	-
0.425 / **40**	30 max.	50 max.	51 max.	-	-	-	-	-	-	-	-	-
0.075 / **200**	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.	36 min.
Characteristics of Materials Passing the #40 Sieve												
Liquid Limit (LL)	-		-	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	41 min.
Plasticity Index (PI)	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.[1]	11 min.[2]
Usual types of significant constituent materials	Stone/gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils		
General rating as subgrade	Excellent to Good							Fair to Poor				

NP = Non plastic

1) *The plasticity index of A-7-5 subgroup is equal to or less than (LL-30)*

2) *The plasticity index of A-7-6 subgroup is greater than (LL-30)*

As there is no upper limit of group index value when using the above stated AASHTO GI equation, Manitoba historically used a modified AASHTO group index formula to limit the range of the GI values and assign a stiffness (resilient modulus) value corresponding to each GI value. The Modified GI values ranged from 0 to 20, where a GI value of 0 indicated a "good" subgrade material and a GI value of 20 indicated a "poor" material for highway construction. However, the key issue with the Modified GI calculation was that all granular soils, A-1-a, A-1-b, A-2-4, A-2-5 and A-3, yield the same (zero) GI value, Accordingly, they were assigned the same resilient modulus value. In addition, the GI or Modified GI does not properly account for the variation of moisture and contaminants (e.g., organics) in soils when assigning the resilient modulus values. The effect of organics, silts and excessive moisture in subgrade soils were accounted for in pavement designs in Manitoba using some adjustments (increase) to the calculated total structural number (SN). The above process was selected based on experience and available knowledge in the past. With the advancement of technology over time, the availability of new testing equipment/tools and new knowledge/experience, a better estimate of resilient modulus of project specific subgrade soil is possible. As such, Manitoba has dropped the use of Modified AASHTO GI and AASHTO GI based stiffness (resilient modulus) estimation process.

Unified Soil Classification System

ASTM D2487*, "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)"*, provides the details of the Unified Soil Classification System (USCS). It is a comprehensive soil evaluation and classification system that provides more insights into soil physical properties and allows for the soil characterization for potential frost susceptibility.

The USCS has been adopted by many highway agencies and recommended for use in pavement design and assessment (TAC 2013). This new Pavement Assessment and Design Manual of Manitoba has mainly adopted the USCS for pavement design and assessment purposes, especially for frost heave consideration in designs and analysis. However, reference to AASHTO soil classification is also provided, where applicable.

Tables 3.0.2 through 3.0.4 show the typical soil classification based on the unified soil classification system (ASTM D 2487) for material passing 75 mm sieve.

Table 3.0.2: Unified Soil Classification System for Coarse-Grained (>50%Retained on #200 Sieve) Soils (Adopted from ASTM D2487)

Preliminary Classification	Primary Composition	Coefficients of Uniformity and Curvature	Properties of Fines (Atterberg Limits)	Class Symbol	Soil Classification
Gravels: % gravel > % sand	Clean Gravels: <5% fines	Cu ≥4.0 and Cc ≥1.0 to ≤3.0	Not applicable	GW	Soil with <15% Sand: Well-graded gravel Soil with ≥15% Sand: Well-graded gravel with sand
		Cu <4.0 and/or Cc <1.0 and/or Cc >3.0		GP	Soil with <15% Sand: Poorly-graded gravel Soil with ≥15% Sand: Poorly-graded gravel with sand
	Gravels with Silt: 5 to 12% silty fines	Cu ≥4.0 and Cc ≥1.0 to ≤3.0	Classifies as ML or MH (plots below “A” line)	GW-GM	Soil with <15% Sand: Well-graded gravel with (Note 1) silt Soil with ≥15% Sand: Well-graded gravel with (Note 1) silt and sand
		Cu <4.0 and/or Cc <1.0 and/or Cc >3.0		GP-GM	Soil with <15% Sand: Poorly-graded gravel with (Note 1) silt Soil with ≥15% Sand: Poorly-graded gravel with (Note 1) silt and sand
	Gravels with Clay or Silty Clay: 5 to 12% clayey fines	Cu ≥4.0 and Cc ≥1.0 to ≤3.0	Classifies as CL, CH or CL-ML (plots on or above “A” line or in hatched area)	GW-GC	Soil with <15% Sand: Well-graded gravel with (Note 1) clay (or silty clay) Soil with ≥15% Sand: Well-graded gravel with (Note 1) clay (or silty clay) and sand
		Cu <4.0 and/or Cc <1.0 and/or Cc >3.0		GP-GC	Soil with <15% Sand: Poorly-graded gravel with (Note 1) clay (or silty clay) Soil with ≥15% Sand: Poorly-graded gravel with (Note 1) clay (or silty clay) and sand
	Gravels with Fines: >12 % fines	Not applicable	Classifies as ML or MH (plots below “A” line)	GM	Soil with <15% Sand: (Note 1) Silty gravel Soil with ≥15% Sand: (Note 1) Silty gravel with sand
		Not applicable	Classifies as CL, CH (plots on or above “A” line)	GC	Soil with <15% Sand: (Note 1) Clayey gravel Soil with ≥15% Sand: (Note 1) Clayey gravel with sand
		Not applicable	Classifies as CL-ML (plots in hatched area)	GC-GM	Soil with <15% Sand: (Note 1) Silty, clayey gravel Soil with ≥15% Sand: (Note 1) Silty, clayey gravel with sand
Sands: % sand > % gravel	Clean Sands: <5% fines	Cu ≥6.0 and Cc ≥1.0 to ≤3.0	Not applicable	SW	Soil with <15% Gravel: Well-graded sand Soil with ≥15% Gravel: Well-graded sand with gravel
		Cu <6.0 and/or Cc <1.0 and/or Cc >3.0		SP	Soil with <15% Gravel: Poorly-graded sand Soil with ≥15% Gravel: Poorly-graded sand with gravel
	Sands with Silt: 5 to 12% silty fines	Cu ≥6.0 and Cc ≥1.0 to ≤3.0	Classifies as ML or MH (plots below “A” line)	SW-SM	Soil with <15% Gravel: Well-graded sand with (Note 1) silt Soil with ≥15% Gravel: Well-graded sand with (Note 1) silt and gravel
		Cu <6.0 and/or Cc <1.0 and/or Cc >3.0		SP-SM	Soil with <15% Gravel: Poorly-graded sand with (Note 1) silt Soil with ≥15% Gravel: Poorly-graded sand with (Note 1) silt and gravel
	Sand with Clay: 5 to 12% clayey fines	Cu ≥6.0 and Cc ≥1.0 to ≤3.0	Classifies as CL, CH or CL-ML (plots on or above “A” line or in hatched area)	SW-SC	Soil with <15% Gravel: Well-graded sand with (Note 1) clay (or silty clay) Soil with ≥15% Gravel: Well-graded sand with (Note 1) clay (or silty clay) and gravel
		Cu <6.0 and/or Cc <1.0 and/or Cc >3.0		SP-SC	Soil with <15% Gravel: Poorly-graded sand with (Note 1) clay (or silty clay) Soil with ≥15% Gravel: Poorly-graded sand with (Note 1) clay (or silty clay) and gravel
	Sands with Fines: >12 % fines	Not applicable	Classifies as ML or MH (plots below “A” line)	SM	Soil with <15% Gravel: (Note 1) Silty sand Soil with ≥15% Gravel: (Note 1) Silty sand with gravel
		Not applicable	Classifies as CL, CH (plots on or above “A” line)	SC	Soil with <15% Gravel: (Note 1) Clayey sand Soil with ≥15% Gravel: (Note 1) Clayey sand with gravel
		Not applicable	Classifies as CL-ML (plots in hatched area)	SC-SM	Soil with <15% Gravel: (Note 1) Silty, clayey sand Soil with ≥15% Gravel: (Note 1) Silty, clayey sand with gravel

Note 1: Add the word “organic” here if the fine material is classified as organic i.e., if $(LL_{ovendried}/LL_{undried})<0.75$

Table 3.0.3: Unified Soil Classification System for Inorganic Fine-Grained (50% or More Passes #200 Sieve) Soils (Adopted from ASTM D2487)

Preliminary Classification	Atterberg Limits	Class Symbol	%Retained on #200 Sieve	Distribution of Coarse Materials	Soil Classification
Inorganic Silts and Clays: Liquid Limit <50	PI <4 or plots below "A" line	ML	<30	<15% retained on #200 sieve	Silt
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Silt with sand % sand < % gravel: Silt with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy silt ≥15% gravel: Sandy silt with gravel
				% sand < % gravel	<15% sand: Gravelly silt ≥15% sand: Gravelly silt with sand
	PI = 4 to 7 and plots on or above "A" line	CL-ML	<30	<15% retained on #200 sieve	Silty Clay
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Silty clay with sand % sand < % gravel: Silty clay with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy silty clay ≥15% gravel: Sandy silty clay with gravel
				% sand < % gravel	<15% sand: Gravelly silty clay ≥15% sand: Gravelly silty clay with sand
	PI >7 and plots on or above "A" line	CL	<30	<15% retained on #200 sieve	Lean Clay
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Lean clay with sand % sand < % gravel: Lean clay with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy lean clay ≥15% gravel: Sandy lean clay with gravel
				% sand < % gravel	<15% sand: Gravelly lean clay ≥15% sand: Gravelly lean clay with sand
Inorganic Silts and Clays: Liquid Limit ≥50	Plots below "A" line	MH	<30	<15% retained on #200 sieve	Elastic silt
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Elastic silt with sand % sand < % gravel: Elastic silt with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy elastic silt ≥15% gravel: Sandy elastic silt with gravel
				% sand < % gravel	<15% sand: Gravelly elastic silt ≥15% sand: Gravelly elastic silt with sand
	PI >7 and plots on or above "A" line	CH	<30	<15% retained on #200 sieve	Fat Clay
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Fat clay with sand % sand < % gravel: Fat clay with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy fat clay ≥15% gravel: Sandy fat clay with gravel
				% sand < % gravel	<15% sand: Gravelly fat clay ≥15% sand: Gravelly fat clay with sand

Table 3.0.4: Unified Soil Classification System for Organic Fine-Grained (50% or More Passes #200 Sieve) Soils (Adopted from ASTM D2487)

Preliminary Classification	Class Symbol	Atterberg Limits	%Retained on #200 Sieve	Distribution of Coarse Materials	Soil Classification
Organic Silts and Clays: Liquid Limit <50	OL	PI <4 or plots below “A” line	<30	<15% retained on #200 sieve	Organic silt
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Organic silt with sand % sand < % gravel: Organic silt with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy organic silt ≥15% gravel: Sandy organic silt with gravel
				% sand < % gravel	<15% sand: Gravelly organic silt ≥15% sand: Gravelly organic silt with sand
		PI ≥4 and plots on or above “A” line	<30	<15% retained on #200 sieve	Organic clay
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Organic clay with sand % sand < % gravel: Organic clay with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy organic clay ≥15% gravel: Sandy organic clay with gravel
				% sand < % gravel	<15% sand: Gravelly organic clay ≥15% sand: Gravelly organic clay with sand
Organic Silts and Clays: Liquid Limit ≥50	OH	PI <4 or plots below “A” line	<30	<15% retained on #200 sieve	Organic silt
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Organic silt with sand % sand < % gravel: Organic silt with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy organic silt ≥15% gravel: Sandy organic silt with gravel
				% sand < % gravel	<15% sand: Gravelly organic silt ≥15% sand: Gravelly organic silt with sand
		Plots on or above “A” line	<30	<15% retained on #200 sieve	Organic clay
				15% to <30% retained on #200 sieve	% sand ≥ % gravel: Organic clay with sand % sand < % gravel: Organic clay with gravel
			≥30	% sand ≥ % gravel	<15% gravel: Sandy organic clay ≥15% gravel: Sandy organic clay with gravel
				% sand < % gravel	<15% sand: Gravelly organic clay ≥15% sand: Gravelly organic clay with sand
Highly Organic Soils	PT	Mainly composed of organic material with dark color and organic odour			Peat

In the unified soil classification system, coarse fraction refers to material passing 75 mm sieve and retained on #200 (0.075 mm) sieve, gravel refers to material passing 75 mm sieve and retained on #4 (4.75 mm) sieve, sand refers to material passing #4 (4.75 mm) sieve and retained on #200 (0.075 mm) sieve, and fines refers to material passing #200 (0.075 mm) sieve. Coarse gravel refers to material that passes 75 mm sieve and retains on 19 mm sieve, while fine gravel refers to material that passes 19 mm sieve and retains on #4 (4.75 mm) sieve. Coarse sand refers to material that passes #4 (4.75 mm) sieve and retains on #10 (2.00 mm) sieve, medium sand refers to material that passes #10 (2.00 mm) sieve and retains on #40 (0.425 mm) sieve, and fine sand refers to material that passes #40 (0.075 mm) sieve and retains on #200 (0.075 mm) sieve. Two gradation parameters, namely coefficient of uniformity (Cu) and coefficient of curvature (Cc), indicate whether a predominantly gravel or sand material is well graded or poorly graded. The Cu and Cc are calculated using the following equations (ASTM D 2487):

$$C_u = \frac{D_{60}}{D_{10}} \tag{3.2}$$

$$C_c = \frac{(D_{30})^2}{D_{60} * D_{10}} \tag{3.3}$$

where,

D_{60} = grain size corresponding to 60% passing, mm
D_{30} = grain size corresponding to 30% passing, mm
D_{10} = grain size corresponding to 10% passing, mm

The liquid limit (LL) and plasticity index (PI) of a soil will indicate whether the soil contains silt, clay or both (refer to Figure 3.0.1 for soil plasticity chart). Clay or clayey soils exhibit plastic behaviour when wet and considerable strength when dry. Silt or silty soils are non-plastic or slightly plastic when wet and exhibit little or no strength when dry. A soil is considered organic when the organic content is high enough to influence the soil properties. The soil is considered have sufficient organic, to call organic soil, if the ratio of liquid limit of the oven dried soil to the liquid limit of the undried soil is less than 0.75 (ASTM D 2487).

In the unified soil class symbols, “L” refers to low liquid limit (<50) and “H” refers to high liquid limit (≥50). The “U” line in the soil plasticity chart (Figure 3.0.1) indicates approximate upper limit of natural moisture content of soils with different plasticity properties. A high liquid limit of a clayey soil generally indicates that the soil is high plastic and expansive, and it has a high swelling potential. Alternatively, the high liquid limit of a silty soil generally indicates that the soil is highly compressible or elastic (not high plastic).

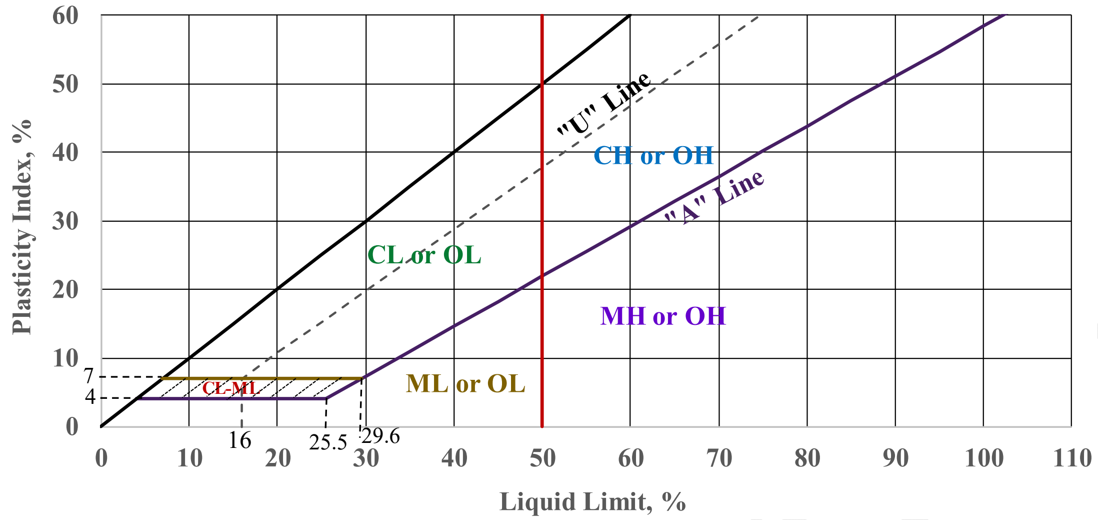

Figure 3.0.1: Soil Plasticity Chart (Adopted from ASTM D2487)

3.3 Subgrade Soils Special Issues and Considerations

Subgrade soils are usually unprocessed native or in-situ materials. Constituents in these soils can vary widely and may cause several performance issues including reduction of subgrade stiffness, non-uniform support and unstable support leading to premature failures of pavement structures. The commonly experienced problematic soils are: frost susceptible, swelling clay and organic soils. Any fine-grained soil with a high moisture content can also cause low stiffness and stability issues.

3.3.1 Frost Susceptible Soils

Subgrade soil frost heaving, and associated pavement distresses and surface roughness (i.e., pavement serviceability loss) are common phenomena in cold climates. Frost heaving occurs due to an increase in volume of the soil-water matrix as water turns into ice during the winter and early spring. Subgrade soils containing silt particles are known to be frost susceptible as they readily allow for the formation of ice lenses when exposed to moisture and freezing temperatures. Other soils such as clay, soil containing organics and peat, which hold a high amount of moisture, may also undergo frost heaving when freezing of soils occurs.

If the subgrade soil and moisture contents or exposure are uniform along a highway/road section, the frost heaving will be fairly uniform throughout that section. This is not likely to create a very detrimental effect to pavement because the entire section will uniformly rise at the same time when freezing occurs and the entire section will uniformly settle at the same time when thawing occurs. However, such an ideal scenario usually does not exist anywhere, and therefore differential frost heave and settlement are observed that produce a non-uniform or rough pavement surface. This is due to the natural variation of soil properties, composition and moisture exposure along a road section.

Three conditions must coincide for frost heave to occur: a frost susceptible soil, a freezing condition, and a moisture source (TAC 2013). In a cold climate like Manitoba, freezing of the subgrade is common by nature. Frost susceptible soils are present in almost all areas of the province. However, it is more prevalent in southwestern Manitoba. Where the above-mentioned all three factors (including moisture exposure) coincide, frost heaving, thawing related settlement, increased road surface roughness and pavement deterioration occurs.

Several guidelines are available for characterizing soils as frost susceptible with recommendations for remedial measures such as the provision of proper drainage, insulation, soil treatment, and soil removal and replacement. A small number of agencies adjust pavement thickness to provide additional overburden and an insulating layer to control or limit frost heaving issues. Removal or treatment of the frost susceptible material are ideal solutions to avoid frost related road surface roughness and pavement deterioration. However, these are generally cost prohibitive and/or an impractical option for most highways in Manitoba. Soil frost susceptibility classification also varies among agencies and published documents, hindering the selection of an appropriate approach to consider soil frost susceptibility in pavement design and assessment.

3.3.2 Subgrade Soil Classification and Frost Susceptibility

In Manitoba, silty or clayey gravel and sand, and silty soils with AASHTO classifications A-2 (A-2-4, A-2-5, A-2-6 and A-2-7), A-4 and A-5 had been historically considered to be potentially frost susceptible subgrade soils. However, when designing pavement structures, a subgrade soil was classified as frost susceptible if it had all the following characteristics (MTGS 2004):

i) quantity of material The passing the 75 µm (No. 200) sieve is 20% or greater;

ii) The plasticity index is 12 or less;

iii) The clay (particles size smaller than 0.005 mm) content is 25% or less;

iv) The combined silt (particle sizes <0.075 mm to 0.005 mm) and fine sand (particle sizes <0.425 mm to 0.075 mm) content is 60% or greater; and

v) The combined coarse sand (particle sizes <2.00 mm to 0.425 mm) and coarse aggregate (retained on 2.00 sieve) is 20% or less.

All organic soils were also considered frost susceptible. However, the above stated soil characterization system categorized soils as frost susceptible or non-frost susceptible without subgrouping into different frost heave severity groups.

The AASHTO pavement design guide (AASHTO 1993) states that a reliable method for recognizing material as frost susceptible for site specific conditions has not yet been identified. However, some guidelines are available in literature. For example, the U.S. Army Corps of Engineers reported that most inorganic soils containing 3% or more particles finer than 0.02 mm size are considered frost susceptible for pavement design purposes (AASHTO 1993, Linell et al. 1963). The frost susceptibility classification and frost heave rates of different soils that were developed by the U.S. Army Corps of Engineers (Kaplan 1974) have been adopted in the AASHTO pavement design guide (AASHTO 1993) to consider frost heave related pavement damage (serviceability loss) in pavement structural design. The frost severity classification varies from negligible to very high depending on the unified soil classification of soils and percentage of material, by weight, smaller than 0.02 mm size.

The TAC PADMG (TAC 2013) presented the nomograph developed by Chamberlain (Chamberlain 1982) for characterizing the frost susceptibility of subgrade soils. Ontario's limiting grain size values for sand, clay and silt is then superimposed on Chamberlain's nomograph to classify soils into acceptable, borderline or unacceptable materials. According to Ontario Pavement Design and Rehabilitation Manual (MTO 2013), fine-grained soils that have high capillarity and low cohesion characteristics are more prone to frost heaving than other soils. The percentage of soil material with grain size between 5.0 and 75 microns is used to classify soils into low (≤40% material in 5.0 and 75 microns particle size range), moderate (40 to 55% material in 5.0 and 75 microns particle size range) and high (55 to 100% material in 5.0 and 75 microns particle size range) frost susceptibility groups.

The FHWA Reference Manual NHI-05-037 (Christopher et al. 2006) specified four conditions that are associated with a high frost hazard potential. These include:

i) presence of a water table within 3.0 m of the pavement surface;

ii) observed frost heaves in the concerned area;

iii) presence of an inorganic soils containing 3% or more, by weight, materials smaller than 0.02 mm; and

iv) potential for the ponding of surface water in pavement structure and subgrade.

The conditions associated with a low frost hazard potential include:

i) a water table greater than 6.0 m below the pavement surface;

ii) low natural moisture content in the frost zone;

iii) embankment surfaces more than 1-2 m above the adjacent grades that provides some insulation and weight to resist frost heaving; and

iv) treatment to eliminate frost issue.

Table 3.0.5 presents several examples of subgrade soils including their gradation, soil index properties and the AASHTO as well as Unified soil classifications. The last four columns show the frost susceptibility classification according to AASHTO (AASHTO 1993), Manitoba (MTGS 2004), TAC (TAC 2013) and Ontario (MTO 2013) guides or manuals for each soil sample. The table shows that soil characterization as frost susceptible and classification into frost susceptibility (severity) groups vary widely among the referenced four guides/manuals. Field observation in Manitoba showed mixed results where some subgrade soils were classified as frost susceptible, as per Manitoba's previous Pavement Design Manual (MTGS 2004), but no frost heaves were experienced. Alternatively, severe frost heaves were experienced in some areas, but subgrade soils were classified as non-frost susceptible following Manitoba's pavement design manual. Furthermore, Manitoba's past frost susceptibility characterization does not distinguish among frost heave severity levels or frost heave rates for various soil classes and compositions. Therefore, a new process was required to better characterize and categorize subgrade soils for frost susceptibility.

A limited investigation of actual field experience of frost heave issues including their severity levels, and laboratory testing and analysis of subgrade soils in Manitoba showed that soil properties and actual frost heave conditions more closely match with the frost severity classification presented in the AASHTO 1993 design guide (AASHTO 1993).

Table 3.0.5: Comparison of Subgrade Soils Frost Susceptibility Classification

Soil Type	% Passing							Soil Properties		Soil Classification				Frost Susceptibility			
Sieve Size	***4.75mm***	***2.00mm***	***0.425mm***	***0.075mm***	***0.05mm***	***0.02mm***	***0.005mm***	**LL, %**	**PI, %**	**AASHTO**	**ASTM/Unified Soil Classification**		**GI**	**AASHTO**	**MB**	**TAC**	**ON**
Fine Sand	74.0	71.0	68.0	25.0	23.0	18.0	11.0	NP	NP	A-2-4	SM	Silty Sand with Gravel	0	High	X	X	Low
Silty Sand	98.0	97.0	93.0	29.0	24.0	19.0	9.0	NP	NP	A-2-4	SM	Silty Sand	0	High	FS	X	Low
Fine Sand	100.0	100.0	100.0	11.0	10.0	7.5	4.0	NP	NP	A-3	SP-SM	Poorly Graded Sand with Silt	0	High	X	X	Low
Gravel	51.0	36.0	21.0	8.0	7.0	6.0	3.0	NP	NP	A-1-a	GW-GM	Well Graded Gravel with Silt and Sand	0	Medium	X	X	Low
Sandy Silt	91.8	89.0	83.0	55.0	50.0	37.0	18.0	16	2	A-4	ML	Sandy Silt	4	High	FS	B	Low
Sandy Silt	91.8	89.0	83.0	56.0	51.0	39.5	22.0	16	6	A-4	CL-ML	Sandy Silty Clay	4	Medium	FS	X	Low
Silt	98.0	97.0	95.0	66.0	60.0	48.0	30.0	17	2	A-4	ML	Sandy Silt	6	Very High	X	X	Low
Silt	100.0	100.0	100.0	99.0	89.0	66.0	32.0	NP	NP	A-4	ML	Silt	8	Very High	X	FS	High
Silt	98.0	97.0	96.0	75.0	66.0	48.0	19.0	16	2	A-4	ML	Silt with Sand	8	Very High	FS	FS	High
Sandy Clay	97.0	96.0	93.0	60.0	56.0	47.0	33.0	34	18	A-6	CL	Sandy Lean Clay	8	Medium	X	X	Low
Low Plastic Clay	99.0	99.0	98.0	67.0	63.0	53.0	38.0	37	20	A-6	CL	Sandy Lean Clay	10	High	X	X	Low
Low Plastic Clay	100.0	100.0	100.0	93.0	84.0	64.0	33.0	32	14	A-6	CL	Lean Clay	10	High	X	FS	High
Low Plastic Clay	98.5	98.0	98.0	75.0	71.0	63.0	52.0	38	20	A-6	CL	Lean Clay with Sand	12	Very High	X	X	Low
Low Plastic Clay	100.0	100.0	100.0	94.0	86.0	69.0	42.0	36	19	A-6	CL	Lean Clay	12	High	X	B	Moderare
Low Plastic Clay	100.0	100.0	100.0	93.0	86.0	70.0	46.0	43	24	A-7-6	CL	Lean Clay	14	High	X	B	Moderare
High Plastic Clay	100.0	100.0	100.0	80.0	74.0	62.0	43.0	45	23	A-7-6	CL	Lean Clay with Sand	14	Very High	X	X	Low
High Plastic Clay	97.0	96.0	95.0	85.0	81.0	74.0	62.0	76	49	A-7-6	CH	Fat Clay with Sand	20	Very Low	X	X	Low
High Plastic Clay	100.0	100.0	100.0	98.0	96.0	92.0	86.0	81	49	A-7-5	CH	Fat Clay	20	Negligible	X	X	Low

LL = Liquid Limit;

PI = Plastic Limit;

NP = Non-Plastic;

AASHTO = American Association of State Highway and Transportation Officials;

ASTM = American Society of Testing and Materials;

TAC = Transportation Association of Canada;

MB = Manitoba; and

ON = Ontario

3.3.3 Consideration of Subgrade Frost Heave into Pavement Design

Until 2016, for new construction and full depth reconstruction projects, Manitoba had been increasing the calculated structural number by 25% if a soil was classified as a frost susceptible. There was no frost heave severity classification in use. The minimum increase in granular base/sub-base thickness was 100 mm. Geotextiles were recommended as a separator between subgrade and subbase, and as a reinforcement for frost susceptible subgrade soils.

The U.S. FHWA Policy Guide (FHWA 1999) for pavement design states that a uniform, stiff as well as moisture and frost resistant foundation is the most important aspect of pavement structural design. The guide recommended stabilizing the upper 300 to 600 mm of fine-grained clay or silt subgrade soils and using 200 to 600 mm thick non-frost susceptible granular subbase layer where frost penetration occurs. The guide also recommended using a free draining base layer underneath the surface layer.

The TAC PADMG (TAC 2013) specified the desirable heights of the top of the subgrade above the high water level (HWL) as a function of the subgrade soil type. Where the maximum water level is known, the subgrade top should be at a minimum of 0.6 m for clean gravel/rock and 1.2 m for silt/clay above the maximum water level.

MTO's pavement design manual (MTO 2013) recommends utilizing uniform subgrade soils, using reduced subgrade soil strength during the spring thaw period, preventing water from entering into the area by providing adequate side ditches and/or sub-drains, and using paved shoulders and/or edge sub-drains to prevent surface water from entering into the subgrade. Other treatments include soil replacement to prevent differential frost heaves and to use expanded or extruded polystyrene.

The engineering and design manual developed by the U.S. Army Corps of Engineers (U.S. Army 1984) provided two alternative approaches to consider subgrade soils frost heave into pavement designs. They are: i) limited subgrade frost penetration method; and ii) reduced subgrade strength method. The first method is meant to control frost heave and associated pavement deterioration by proving an adequate thickness of pavement structure (surface, base and subbase). This will limit the penetration of frost into the frost-susceptible subgrade. The manual also states that the prevention of frost penetration into the subgrade soils is uneconomical in almost all scenarios and unnecessary. The manual recommends that the limited subgrade soil frost penetration method should be only used in locations where less thickness than the reduced subgrade strength method is required.

The reduced subgrade strength method determines the thickness of pavement structures that are required to carry the design traffic loads considering occasional subgrade weakening due to frost melting. This method is applicable to a road section where the subgrade soil composition and moisture exposure are uniform throughout the section. This approach is not suitable where differential heaving due to frost penetration and differential settlement due to frost melting occur (U.S. Army 1984). As mentioned in the previous section of this manual, such an ideal condition (uniform frost heave and settlement) rarely exists in the field.

The use of reduced subgrade strength for subgrade melting and weakening in spring season is a standard practice for pavement design in cold climates where spring thaw weakening is a normal phenomenon for all pavements and subgrade soils. For example, Manitoba uses a low subgrade resilient modulus value for the spring season, which is equivalent to 25% to 50% of the summer/fall modulus values, to determine the effective (annual representative) subgrade resilient modulus or support value. The pavement structure based on this reduced subgrade strength is adequate to carry traffic loads during the critical spring thawing period. This method may not be adequate to address the differential frost heaving and thawing, and the associated road surface roughness or serviceability loss, as experienced in Manitoba. The reason is that any additional subgrade weakening due to frost melting, which is specific to the frost susceptible soils, may not be fully accounted for by the general 50% or 75% reduction for spring condition. A research project to investigate/analyze this issue is currently underway in Manitoba. Any changes to subgrade inputs in pavement design resulting from the new study will be reflected in the latest version of the department’s relevant engineering standard.

FHWA Reference Manual NHI-05-037 (Christopher et al. 2006) recommended several alternatives to improve subgrade when frost-susceptible soils are encountered, which include the following:

i) Remove the frost susceptible soils that are in Groups F3 and F4 (see Figure 3.0.2) and replace with non-frost susceptible material(s) up to the depth of frost penetration;

ii) Place non-frost susceptible materials of a thickness that prevent freezing of frost susceptible soils that are in Groups F2, F3 and F4 (see Figure 3.0.2);

iii) Remove isolated pockets of frost-susceptible soils to eliminate abrupt changes in subgrade conditions;

iv) Stabilize the frost-susceptible soil by: a) mechanically removing fines or treating with cementitious materials, b) reducing/preventing moisture migration, or c) altering the freezing point of the soil moisture; and

v) Increase the pavement thickness to account for the subgrade strength reduction during the spring-thawing period for frost-susceptible soils that are in Groups F1, F2 and F3 (see Figure 3.0.2).

The AASHTO Design Guide (AASHTO 1993) states that it is feasible to control frost heave by increasing the thickness of pavement structure with non-frost susceptible materials. The most acceptable practice is to remove the frost susceptible materials and replace them with non-frost susceptible materials to a depth of one-half of the frost depth. If frost mitigation measure has been taken, the serviceability loss due to frost heave should be ignored in pavement design.

The AASHTO 1993 guide does not recommend increasing pavement thickness for frost heave issues because a small increase in pavement thickness has minimal impact in reducing or eliminating the frost heave issue. It recommends a pavement design for reduced service life considering the serviceability losses due to frost heave and the traffic loads. If frost heave is to be considered in the design, in terms of serviceability loss due to frost, the guide recommends an approach similar to stage construction. However, the outlined approach in the AASHTO 1993 guide can be used to increase pavement structure for a higher service life of the initial pavement considering the estimated serviceability loss due to the frost heave and the serviceability loss available for traffic loads, if desired.

In the absence of any other better approach, the AASHTO 1993 guide approach is considered more appropriate for cost-effective pavement design and construction in Manitoba. The general practice now should be frost heave management, not frost heave protection i.e., no increase in pavement thickness for all rehabilitation and the partial depth reconstruction designs where the design subgrade elevation remains unchanged. For new construction and the full depth reconstruction, where the pavement surface elevation could be raised or the design subgrade elevation could be lowered, consideration should be given to accommodate additional granular subbase and/or fill materials depending on the degree of frost issues, frost penetration depth, importance of the highway, available budgets and availability of materials. However, the initial pavement service life should, in no case, be less than 10 years.

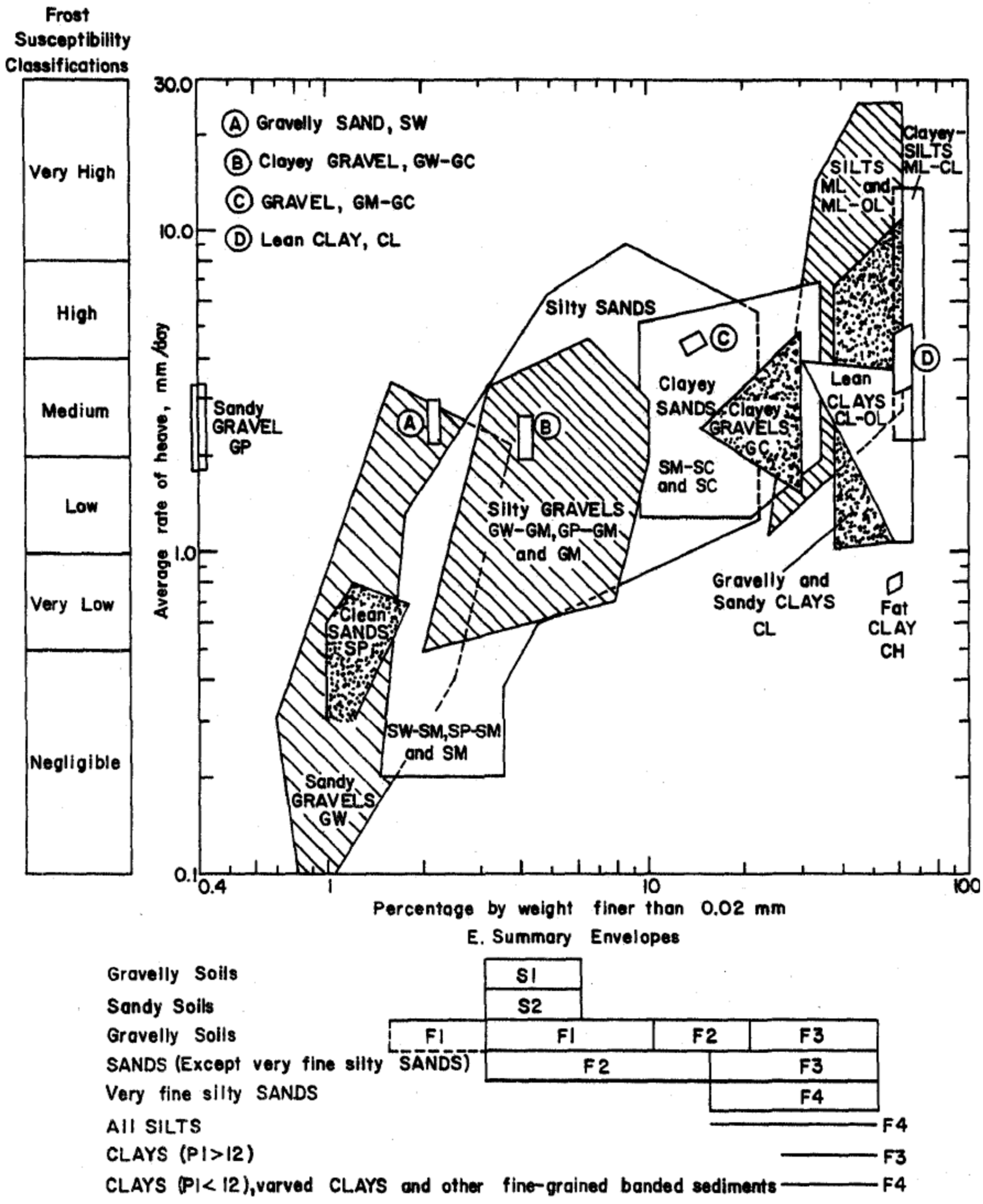

Figure 3.0.2: Subgrade Soil Frost Classification and Frost Heave Rate (U.S. Army 1984)

The use of non-frost susceptible granular fill, instead of the excavated in-situ frost susceptible material, in subgrade sub-cut (typically 0.6 m) is a prudent approach to avoid wider grade with thicker pavement structures. Consideration should also be given to accommodate additional granular subbase and/or non-frost susceptible granular fill material so that the total thickness of pavement structure, including any additional non-frost susceptible granular subbase and/or fill materials, equates to at least 50% of the frost penetration depth at the project site if the frost issue is severe to very severe, except for the National Highway System (NHS) Core and Intermodal routes, which include PTH 1, PTH 16, PTH 75, PTH 100, PTH 101 and PTH 190. For these NHS Core and Intermodal routes, the total thickness of a pavement structure, including any additional non-frost susceptible granular subbase and/or fill materials, should equate to at least 70% of the frost penetration depth at the project site if the frost issue is severe to very severe.

3.3.4 Swelling Soils

Soils (e.g., certain lacustrine clays) containing very active minerals such as bentonite and montmorillonite possess swelling and shrinking properties. Such subgrade soils can cause severe distress in pavement layers and surface, especially in areas where seasonal moisture fluctuation occurs (TAC 2013). Expansive or swelling subgrade soils are common in and around the Winnipeg area and they also may exist in other areas elsewhere in Manitoba.

With a seasonal increase of moisture content exceeding the saturation moisture content, swelling of embankment soils can occur. Such swelling will then cause expansion of a highway embankment, predominantly in unconfined and less confined directions. The swelling is generally proceeded by shrinkage or contraction (in all directions) of embankment as excess moisture dissipates from the saturated or oversaturated soils. This swelling or expansion and shrinkage or contraction activities can result in separation or cracking and settlement in an embankment, which are then manifested as longitudinal cracking and transverse cracking on pavement surface and collapse of a part of the pavement structure (e.g., along centreline or lane edge). Differential swelling and shrinkage (expansion and contraction) due to the variation of soil composition, consolidation, overburden and moisture exposure can aggravate the issues. The transition between an existing grade and new grade (e.g., in a grade widening project) is an example area for potential variation in material composition and consolidation.

Potential solutions to eliminate or minimize swelling issues in subgrade/embankment are sub-excavation to remove expansive soil and replacement with non-expansive soil (preferably select granular material), application of adequate overburden on expansive soils to neutralize the swelling pressure, building embankment with granular materials and soil stabilization with cementitious materials (portland cement, lime, etc.). Many other alternative solutions and treatments are available that have been applied in various jurisdictions. Experienced geotechnical and pavement engineering professionals should be consulted when building embankment on or with soils that have the potential to create swelling issues in a specific location of a highway to develop site specific cost-effective solution.

3.3.5 Organics in Soils and Organic/Peat Materials

Soils with significant quantities of organics and organic/peat materials including topsoil are undesirable because these materials are susceptible to high compression and shrinkage, settlement, instability and frost heave. They also exhibit low density and strength or stiffness. A proper treatment of organic materials, which occur as topsoil, peat deposits in swamps or muskegs, and organic deposits, is essential for the satisfactory performance of a pavement structure and surface. These materials should be avoided whenever possible. Since highly organic soils are extremely compressible and weak, organics in isolated areas should be removed prior to the construction of an embankment or subgrade. The typical organic contents in subgrade soils in Manitoba are presented in Table 3.0.6. Where applicable, these values can be used as a guideline to estimate the resilient modulus of subgrade soils or correct the design structural number for preliminary design of pavement structures in the absence of soil survey and test data.

Table 3.0.6: Typical Organic Contents of Native Subgrade Soils (MTGS 2004)

Subgrade Soil Types	Organic Contents, %
Light (Sandy) Topsoil	1 - 5
Medium (Silty/Clayey) Topsoil	2 - 10
Heavy (Clay) Topsoil	4 - 8
Organic Silts and Sands	3 - 17
Organic Clays	4 - 11
Peat	17 - 50

The department's "*Guidelines for Handling of Organic Materials*" provide the criteria to determine when organic materials must be removed as waste excavation and the appropriate alternatives to dispose/use of such materials. The designer should utilize the latest version of this guidelines to determine which organic layers must be designated as waste excavation based on the proposed grade-line design, soils information and site conditions.

The waste excavation consists of excavating and disposing of unsuitable material from the right-of-way limits. Once a material has been designated waste, it should not be used as embankment material in any level of the highway embankment. If the removal and replacement of a highly organic soil is not possible (e.g., a thick deposit), it should be bridged with suitable fill materials, including the placement of appropriate geosynthetic material, to ensure long term stability of the embankment. A low stiffness (resilient modulus) value should be assigned (based on the test results on representative soil samples) to a subgrade soil that contains organics and remains in place within the stressed zone under the pavement structure. A suitable correction can be applied to pavement layer thickness in preliminary design, depending on the organic content (%), thickness of soil strata (deposit) that contains organics and depth of that layer from the design subgrade elevation.

All borrow materials for highway embankment construction and reconstruction should be free of visible organics, silts and other unsuitable materials. The organic contents should not exceed 1% in sandy and gravelly soils, and 3% in clayey soils based on laboratory testing on representative soil samples. If the availability of suitable borrow material, including the granular fill, for embankment construction or reconstruction becomes an issue, an inferior quality borrowed clayey, sandy and gravelly soils can be used provided that weighted average organic contents in materials placed in any depth does not exceed 6% with no more than 10% test results exceeding this value. If a borrow source or part of the borrow source does meet above requirements, it should not be used for embankment construction.

The resilient modulus (M_R) or CBR values of borrow material should be determined in the laboratory by testing on representative soil samples. The representative M_R or soaked CBR value at 90% confidence level i.e., the lowest 10^{th} percentile value from the cumulative distribution of all test results from soil samples collected from a borrow source or a designated portion of a borrow source should be determined. If the representative M_R value is less than a specified minimum value (for example, 15 MPa), the borrow source or part of the borrow source with such low representative M_R value should not be used for embankment construction. The pavement structures should be increased based on pavement layer design using the representative M_R value of the acceptable borrow materials. All borrow sources should be preapproved by the department before bringing the borrow materials to the project sites.

If the in-situ subgrade soils provide a lower representative resilient modulus value than that of the overlaying embankment (e.g., borrowed) soils, representative M_R value of the in-situ soils should be used in the design. Alternatively, if the borrowed embankment soils provide a lower representative resilient modulus value than that of the underlaying in-situ subgrade soils, representative M_R value of the borrowed soils should be used in the design. However, if the height of the new embankment (from prairie level to the design subgrade elevation) with borrowed soils is ≥1.0 m, weighted average resilient modulus of various soil layers to a depth up to 3.0 m from the design subgrade elevation can be used considering variation of stress intensity at different depths below the design subgrade level. The adequacy of the pavement structures at transitions between an existing pavement and the new embankment/pavement should be ensured in all cases.

3.4 Granular Subbase Course Materials

The term "subbase" refers to the first layer of processed non-frost susceptible aggregate materials placed on a prepared subgrade or embankment. It transfers the stress of the imposed traffic loads from overlying base and surface layers of a pavement structure to the underlying supporting subgrade (foundation) soils. A subbase material should be stable, reasonably stiff and graded to ensure quick drainage of moisture. The stiffness and stability characteristics of granular subbase materials can vary depending on the mineral properties (composition, hardness, surface texture, density, strength, porosity, etc.), particle size distribution (gradation), index properties (specific gravity, water absorption, liquid limit, plasticity index, etc.), production quality (amounts of fractured, angular, flat and elongated particles, clay lumps, soft/friable particles, etc.) and construction (material uniformity, density, etc.).

Manitoba has historically used Granular C base aggregate material for subbase layer construction. Several other subbase materials such as Modified C base, granular fill (a sandy material), and 50 mm, 75 mm, 100 mm, 125 mm and 150 mm minus crushed rock materials have also been used. The specifications for these alternative materials (other than the Granular C base) varied widely among regions and projects. None of these subbase materials, including Granular C base, are currently in use in Manitoba. However, to assign appropriate structural layer coefficients or moduli values to the in-situ materials, when required, it is important to collect representative samples and test them in the laboratory for physical and stiffness properties. The gradations and other physical properties of Granular C base and typical Modified C base materials are presented in Table 3.0.7.

Table 3.0.7: Physical Requirements for Previously Used Granular Subbase Materials

Passing Sieve Sizes/Other Properties	**Percentage Passing Standard Sieves and Other Properties**		
	C Base		**Modified C Base**
	Gravel	**Limestone**	
50 mm			100%
37.5 mm	100 %	-	
25 mm	85 - 100 %	100 %	
4.75 mm	25 - 80%	25 - 80%	25 - 85%
0.425 mm	15 - 40%		
0.075 mm	8 - 18%	8 - 20%	5 - 18%
Particles with Fractured Faces, Minimum (Retained on 4.75 mm Sieve)	15%	100%	
Shale Content, Maximum (Retained on 4.75 mm Sieve)	20%	-	
L.A. Abrasion Loss, Maximum	40%	40%	
Oversize Particles, Maximum (Not larger than 3.0 mm from the maximum size)	3%	3%	3%

In 2019, the department developed new specifications for granular subbase materials and started to use them in the 2020 construction season. The new subbase materials, which are classified into several types to meet various local requirements and materials availability, include:

Granular Subbase Class C (GSB- C): A good quality granular subbase material for use below the granular base layer.

Granular Subbase Class F (GSB- F): A fair quality granular subbase material for use as a fill below the granular base or GSB- C layer.

Crushed Rock Minus 50 mm (CR- M50): A premium quality granular subbase material for use below the granular base layer.

Crushed Rock Minus 100 mm (CR- M100): A high quality granular subbase or fill material for use below the CR- M50 layer.

Crushed Rock Minus 125 mm (CR- M125): A good quality granular subbase or fill material for use below the CR- M50 or CR-M100 layer.

The physical requirements of the various subbase materials are presented in Table 3.0.8.

Table 3.0.8: Physical Requirements for New Subbase Course Materials

Passing Sieve Size (Note 1)		GSB- C		GSB- F		CR- M50		CR- M100		CR- M125	
Metric, mm	**Imperial**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**
125.00	5"									100	100
100.000	4"							100	100		
75.000	3"			100	100			60	100	55	90
50.000	2"					100	100				
37.500	1 1/2"	100	100	75	100	65	100	35	80	30	70
25.000	1"										
19.000	3/4"	70	100	55	100	40	75	20	60	15	55
16.000	5/8"										
12.500	1/2"										
9.500	3/8"	50	95	40	100	25	55	15	45	10	40
4.750	#4	35	80	30	90	15	40	10	35		
2.000	#10	25	60	20	70	10	30				
0.850	#20										
0.425	#40										
0.180	#80										
0.075	#200	5	12	5	15	0	8	0	8	0	8
Fractured Faces, Min. %		20		N/A		100%		100%		100%	
Plasticity Index, Max. %		6		6		NP		N/A		N/A	
Liquid Limit, Max. %		25		25		25		N/A		N/A	
L.A. Abrasion Loss, Max. %		40 (ASTM C 131)		40 (ASTM C 131)		40 (ASTM C 535)		40 (ASTM C 535)		40 (ASTM C 535)	
Lightweight Particles Content, Max. %		12		12		12		12		12	
Clay Lumps and Friable Particles Content in Materials Retained on 2.36 mm (#16 Sieve), Max. %		3.0		3.0		2.0		2.0		2.0	
Bulk Specific Gravity (Oven Dry Basis) of Coarse and Fine Aggregates, Min. (Note 2)		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design	
Water Absorption of Coarse and Fine Aggregates, Max. % (Note 2)		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design	

Note 1: A maximum of three percentage (3%) oversize particles will be allowed provided that the maximum dimension of the oversize particles does not exceed 3.0 mm from the specified maximum size.

Note 2: Coarse aggregate refers to material retained on 4.75 mm (#4) sieve; fine aggregate refers to material passing 4.75 mm (#4) sieve and retained on 0.075 mm (#200 sieve).

The aggregates for subbase should exhibit a minimum bulk specific gravity (over dry basis) of 2.50 (preferably ≥2.60) and a maximum water absorption of 3.5% (preferably ≤2.5%). Subbase material exhibiting a specific gravity of less than 2.50 and/or water absorption of more than

3.5% can be used if no suitable alternative source is available within a reasonable distance from the project site. In that case, the subbase material should be tested for resilient modulus or soaked CBR value to assign appropriate structural layer coefficient value and adjust the layer thickness of pavement material(s). The quality requirements of aggregates for subbase materials, corresponding to the structural layer coefficient or resilient moduli values used in the pavement design, should be specified in the construction tender and design-build contract's technical requirements.

If CR-M100 or CR- M125 is used as a subbase or fill material, it should be overlaid with a layer of CR- M50 material before placing the granular base layer material to avoid drain down of fine aggregates from granular base layer into the CR-M100/CR- M125 layer. If no CR- M50 material is available (e.g., not feasible to produce because of small quantity), a heavy-duty geosynthetic fabric should be placed on the top of compacted CR- M100/CR- M125 layer before placing the granular base layer.

3.5 Granular Base Course Materials

The granular base course is a layer of granular aggregate material placed below the AC, portland cement concrete and AST surface layers or placed as a top layer of unpaved roads and shoulders (gravel roads and shoulders). The granular base materials are processed non-frost susceptible aggregates and generally better-quality materials than the underlying subbase layer materials. The base layer transfers the stress of the imposed traffic loads from overlying surface layers of a pavement structure to the underlying subbase layer(s) and supporting subgrade (foundation). A base course material should be stable, stiff, durable and hard to withstand stress from traffic load and environmental exposure, and properly graded to ensure quick drainage of moisture. The stiffness and stability characteristics of granular base materials can also vary depending on the mineral properties (composition, hardness, surface texture, density, strength, porosity, etc.), gradations, index properties (specific gravity, water absorption, liquid limit, plasticity index, etc.), production quality (amounts of fractured, angular, flat and elongated particles, clay lumps, soft and friable particles, etc.) and placement (material uniformity, density, etc.).

Manitoba has historically used Granular A base as a base course material and several specifications for traffic gravel material. As in the case of old Granular C and other alternative subbase materials, Manitoba is not currently using the old Granular A base material. However, to assign appropriate structural layer coefficient or moduli values to the in-situ materials, when required, it is important to collect representative samples and test them in the laboratory for physical and stiffness properties. The gradations and physical properties of Granular A base materials are presented in Table 3.0.9.

Table 3.0.9: Physical Requirements for Previously Used "Granular A" Base Materials

Passing Sieve Sizes/ Other Properties	Percentage Passing Standard Sieves and Other Properties	
	Gravel	**Limestone**
19.0 mm	100%	100%
16.0 mm	80 - 100%	
4.75 mm	40 - 70% (Average 65% maximum)	35 - 70%
2.00 mm	25 - 55%	
0.425 mm	15 - 30%	10 - 30%
0.075 mm	8 - 15%	8 - 17%
Particles with Fractured Faces, Minimum (Retained on 4.75 mm Sieve)	35%	100%
Shale Content, Maximum (Retained on 4.75 mm Sieve)	12%	0
L.A. Abrasion Loss, Maximum	35%	35%
Clay Balls Content, Maximum (Retained on 12.5 mm sieve)	10%	0
Plasticity Index, Maximum	6%	NP
Oversize Particles, Maximum (Not larger than 22mm)	3%	3%

Manitoba used several trial and interim specifications of the proposed new granular base between 2016 and 2019 with different naming conventions such as DSB-2016 (drainable stable base trial in 2016), DSB-2017, DSB-2018 and Modified A base. Based on the results of trials in the laboratory and field for stability, stiffness, drainage quality, ease of production and construction, and limitation in the quality of available aggregate sources around the province, the department has developed new general specifications for several granular base materials in 2019 and started to use them in 2020 construction season with complete discontinuation of the previously used Granular A base specifications. The new granular base materials, which are also classified into several types to meet various local requirements and materials availability, include the following:

Granular Base Course Type I (GBC- I)**:** A granular base course material of premium quality, with an excellent balance of drainage, stability and stiffness characteristics, for use below the AC, portland cement concrete (PCC), AST and granular surface layers. The fines content (material passing the 0.075 mm sieve) should be limited to 2 to 6% and the materials passing #40 sieve should be non plastic (NP) when using under a PCC layer or as a sandwich layer.

Granular Base Course Type II (GBC- II): A high-quality granular base course material, with a very good balance of drainage, stability and stiffness characteristics, for use below the AC, portland cement concrete (PCC), AST and granular surface layers. It can also be used on the unpaved shoulder surface if fines content is close to the upper limit. The fines content (material passing the 0.075 mm sieve) should be limited to 2 to 6% and the materials passing #40 sieve should be non-plastic (NP) when using under a PCC layer or as a sandwich layer.

Granular Base Course- Modified (GBC- M): A granular base course material, with a good balance of drainage, stability and stiffness characteristics, for use below the AC, AST and granular surface layers. It can also be used on the unpaved shoulder surface if fines content is close to the upper limit.

Granular Base Course- Surface (GBC- S): A granular base course material with low permeability characteristics, for use as granular surface layer material for unpaved shoulders and gravel roads.

The gradations and other physical requirements of the various base course materials are presented in Table 3.0.10. The aggregates for base course should exhibit a minimum bulk specific gravity (over dry basis) of 2.50 (preferably ≥2.60) and a maximum water absorption of 3.5% (preferably ≤2.5%). GBC material exhibiting a specific gravity of less than 2.50 and/or water absorption of more than 3.5% can be used if no suitable alternative source is available within a reasonable distance from the project site. In that case, the GBC material should be tested for resilient modulus or soaked CBR value to assign appropriate structural layer coefficient value and adjust the thickness of pavement layer(s). The quality requirements of aggregates for base materials, corresponding to the structural layer coefficient or resilient moduli values used in the pavement design, should be specified in the construction tender and design-build project’s technical requirements.

Table 3.0.10: Physical Requirements for New Base Course Materials

Passing Sieve Size (Note 1)		GBC- I		GBC- II		GBC- M		GBC-S	
Metric, mm	**Imperial**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**	**Lower Limit**	**Upper Limit**
25.000	1"	100	100						
19.000	3/4"	80	95	100	100	100	100	100	
16.000	5/8"	70	90	80	95	83	100	85	100
12.500	1/2"	55	83	70	90	70	95	70	95
9.500	3/8"	47	75	60	84	60	87	60	88
4.750	#4	33	60	40	66	40	70	40	70
2.000	#10	20	45	24	48	25	50	25	50
0.850	#20	11	30	14	33	15	35	17	38
0.425	#40	7	21	9	24	10	25	12	30
0.180	#80	5	14	6	16	6	17	8	20
0.075	#200	3 (Note 2)	8 (Note 2)	3 (Note 2)	8 (Note 2)	4	9	6	13
Fractured Faces, Min. %		55		55		40		35	
Plasticity Index, Max. %		3		3		3		6	
Liquid Limit, Max. %		25		25		25		25	
L.A. Abrasion Loss, Max. %		35 (ASTM C 131)		35 (ASTM C 131)		35 (ASTM C 131)		35 (ASTM C 131)	
Lightweight Particles Content, Max. %		7		7		7		12	
Clay Lumps and Friable Particles Content in Materials Retained on 2.36 mm (#16 Sieve), Max. %		2.0		2.0		2.0		3.0	
Bulk Specific Gravity (Oven Dry Basis) of Coarse and Fine Aggregates, Min. (Note 3)		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design	
Water Absorption of Coarse and Fine Aggregates, Max. % (Note 3)		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design		As specified in the Pavement Design	

Note 1: A maximum of three percentage (3%) oversize particles will be allowed provided that the maximum dimension of the oversize particles does not exceed 3.0 mm from the specified maximum size.

Note 2: Only GBC Type I or Type II can be used below the PCC layer. The fine material content (material passing the 0.075 mm sieve) should be limited to 2 to 6% and the materials passing #40 sieve should be non-plastic (NP) for such application.

Note 3: Coarse aggregate refers to material retained on 4.75 mm (#4) sieve; fine aggregate refers to material passing 4.75 mm (#4) sieve and retained on 0.075 mm (#200 sieve).

3.6 Treated Subgrade, Subbase and Base Materials

Manitoba uses lime and cement treated (modified or stabilized) subgrade in some locations. Lime is also being used on some projects as a soil modifying agent to reduce moisture content i.e., to improve workability and expedite construction. A cement stabilized Granular A base material was placed in 2015 on PR 330 as a trial basis.

The physical and mechanical properties of treated materials can vary widely depending on the type and composition of the material to be treated, and the type (lime, cement, emulsion, asphalt, etc.) and amount of binder to be used. The selected untreated material must be characterized through laboratory testing to determine the appropriate binder type and required binder content, followed by mix design and full characterization of the treated material to determine design binder content and to assign appropriate stiffness and structural layer coefficient values for pavement design purposes.

The open graded drainage layer (OGDL) is a permeable granular base layer, with a porosity in the range of 0.25 to 0.40, placed between an overlying PCC or AC and an underlying PCC, AC, base or subbase layer to allow for the quick drainage of water from the pavement structure. It can be cement treated, asphalt treated, or crushed/untreated granular material of a uniform aggregate particle size or select gradation to ensure excellent drainage and stability properties.

In Manitoba, the first OGDL (treated with asphalt binder) was placed on PTH 75 under a new PCC pavement on a trial basis in 2009. In 2020, OGDL has been placed as a separator layer between a composite pavement and unbound PCC overlay on a section of PTH 59 (South of Perimeter Highway at the Floodway Bridge).

3.7 Reclaimed and Recycled Materials

The management and disposal of various reclaimed materials and by-products from industrial processing and production are global challenges (TAC 2013). Interest in reusing and recycling of these materials in road/highway construction projects have increased due to a shortage of landfill area, good value of some materials and depletion of natural sources for virgin materials. Manitoba has been using reclaimed asphalt pavement (RAP) as a granular base (e.g., pulverized in-place and re-laid) and partial replacement of virgin aggregates, which reduces virgin asphalt binder as well, in new AC mixes for many years. Recycled concrete has been used as fill or subbase materials on a limited basis. The first trial use of asphalt shingles in AC mixes together with RAP was done on Brady Road (Winnipeg) in 2020. Other forms of recycling the existing

AC pavement in Manitoba are cold in-place recycling (treated with asphalt cement or emulsion), cold-central plant recycling (treated with emulsion), and reclaiming from an existing road, hauling to another area within the project or to another road and laying as RAP surface, base or subbase material.

Several other materials and processes such as crumb rubber in asphalt, crushed glass in aggregate subbase, blast furnace slags in PCC mixes, fly ash in PCC mixes and full depth reclamation (FDR) of asphalt pavements have been used or are still in use elsewhere in Canada and the United States. Among these materials and processes, Manitoba currently uses only the fly ash in all PCC mixes, except for cold weather paving.

The constructability and long-term performance of these reclaimed, reused and recycled materials are concerns in every jurisdiction. Proper processing, characterization, design and use to these materials are critical to assign appropriate structural values, ease production and placement, and to ensure long term performance.

The pavement and project designers should consider following application of existing (in-place) and reclaimed materials from the roadways, as applicable:

1) Reclaimed Asphalt Pavement (RAP) and Existing (In-place) Asphalt: Incorporate good quality RAP into AC mixes, Cold In-place Recycle (CIR), Full Depth Reclamation (FDR) and recycle in-place, Cold Central Plant Recycling (CCPR) of stockpiled RAP, pulverize existing asphalt in-place and relay, use RAP as a subbase and, where applicable, use RAP as a granular fill/embankment with proper structural value to avoid waste or long-term stockpiling.

   Any RAP used as a subbase should be placed below the crushed rock layer (e.g., CR-M50, CR-M100 and CR-M125) or above the GSB-C/GSB-F layer, as applicable. Any RAP used as a fill/embankment material should be placed below the crushed rock or above the native/borrowed (e.g., clay) subgrade/embankment material, as applicable.

2) Existing (In-place) and Reclaimed Concrete: Rubblize in-place or recycle the good quality reclaimed concrete from roadways to produce recycled concrete aggregate (RCA). Use the RCA as subbase and/or subgrade/embankment material, as specified below.

The RCA meeting the CR-M50 granular subbase specifications can be used as a subbase on roads/highways with design ESALs of less than 1,000,000. The RCA meeting the specifications for GBC-I, GBC-II or GBC-M can be used as a subbase on roads/highways with design ESALs of less than 10.0 million. All these RCA materials should exhibit a minimum resilient modulus of 150 MPa or a minimum soaked California Bearing Ratio (CBR) value of 35% and should be given a structural layer coefficient value of 0.12, until further update following extensive testing and evaluation. The RCA meeting the specifications for GBC-I, GBC-II, GBC-M or CR-M50 can be used as a subbase on all AT paths. Any RCA used as a subbase material should be placed below the granular aggregate (e.g., GSB-C, GSB-F, CR-M50, CR-M100 and CR-M125) and RAP subbase layer.

Graded RCA with 100% passing 50 mm sieve, but not more than 70% passing 4.75 mm (#4) sieve and not more than 15% passing 0.075 mm (#200) sieve can be used as a subgrade/embankment material, instead of borrowed soil (e.g., clay) material, on roads/highways with design ESALs of less than 3.0 million and on all AT paths. Any RCA used as a subgrade/embankment material should be placed above the native/borrowed (e.g., clay) soil material. RCA material used as a subgrade/embankment material should exhibit a resilient modulus or soaked California Bearing Ratio (CBR) value which is 50% higher than the underlying native/borrowed (e.g., clay) soil material because of potential degradation of RCA material over time.

The RCA can be blended with virgin granular aggregate material(s) to meet the specifications and/or workability requirements, as applicable. RCA embankment slopes should be properly capped with non-erodible material (e.g., cohesive clay) to avoid washing out of fines.

No RCA should be placed within 300 mm of pavement surface. No RCA material should be placed below the seasonal high-water table elevation. RCA material should also meet applicable environmental requirements and any restrictions for use near metal and PCC structures.

3) Existing Chip Seal Surface (AST): Mill and relay in place or reclaim and use as subbase or fill below the virgin aggregate subbase layer.

4) Salvaged Granular Materials: Salvaged granular materials like A base, C base, GBC-S, GSB-F and other sandy granular fill, which are free from contaminants like silt, organics and other foreign objects, can be used as subgrade/embankment materials. These materials should be placed above the native/borrowed (e.g., clay) soil or below the granular aggregate (e.g., crushed rock) subgrade/embankment material, as applicable.

   Salvaged granular materials like GBC-I, GBC-II, GBC-M, GSB-C, CR-M50, CR-M100 and CR-M125, which are free from contaminants like silt, organics and other foreign objects, can be used as subbase materials with appropriate structural values.

5) Salvaged Embankment Materials: Salvaged embankment soils (e.g., clay), which are free from contaminants like silt, organics, soft/spongy materials and other foreign objects, can be used as new subgrade/embankment materials with appropriate design resilient modulus value for pavement structure or can be used on road slopes.

3.8 Asphalt Binder Materials

3.8.1 Asphalt Binder

Asphalt binder, also called asphalt cement, is a dark brown to black cementitious material in which the predominating constituents are bitumen. Asphalt cement may occur in nature or be obtained in the crude petroleum refining process. Asphalt cement, which is a semisolid to solid material, gradually liquefies when heated and is used for most paving projects on Manitoba provincial highways. Manitoba has historically used the penetration-viscosity grade asphalt binder for AC construction projects. This grading system is based on the hardness, as determined through a standard penetration test at a specific temperature (25°C), and the viscosity of the asphalt binder. A low penetration number indicates a stiffer or harder binder that is to be used to resist rutting due to high traffic loads. Alternatively, a high penetration value indicates a softer binder which is to be used in cold climate with low traffic loads. For example, Manitoba had been using 120-150 penetration grade binder for freeways, expressways (high traffic loads) and 150-200 penetration grade for other highways (moderate to low traffic loads). For some projects in northern Manitoba, with colder climate than southern Manitoba, 200-300 penetration grade asphalt binder was used to limit thermal cracking.

The main issue with the penetration grading system is that there is no balance between the stiffness (hardness), which is required to resist rutting at high temperatures and different design traffic loads, and the flexibility, which is required to resist fatigue cracking and thermal cracking at intermediate and low temperatures under different environmental exposures. That is why many North American jurisdictions, including Manitoba, have experienced thermal cracking as the most predominant distress in AC pavement followed by rutting. Rutting is predominant on highways/roads with high traffic loads in terms of total number of truck traffic or load repetitions, creeping loads due to slow speed and impact loads due to stop (e.g., at intersections). The other issue with this grading system is that it does not account for the impact of short-term age hardening during mix production and transportation, and the long-term age hardening of asphalt binder while the AC pavements are in service. To address these shortcomings, a new binder grading system, called the SuperPave performance grade (PG), was developed for asphalt binder under the U.S. Strategic Highway Research Program (SHRP).

The PG specification classifies asphalt binders into distinct grading based on the performance requirements at both high and low seasonal temperatures that an AC pavement will experience. The selection of PG binder is project specific based on the project environmental exposure as the binder must comply with both the low and high pavement temperatures while it is in service. For example, a PG 58-40 asphalt binder must meet the performance requirements at a high pavement temperature of 58°C (the highest seven-day average of the daily maximum temperatures, measured at 20 mm depth below the pavement surface) to resist rutting under traffic load as well as at a low pavement temperature of -40°C (the lowest daily temperature, measured at pavement surface) to resist thermal cracking. The high temperature grade must be bumped up for high traffic loads, slow traffic loads and stop (intersection) conditions to resist pavement from the potential higher rutting in these scenarios. Manitoba started the use of PG asphalt binder a decade ago on a trial basis with a full implementation of PG asphalt binder and complete discontinuation of penetration grade binder in 2018.

The testing protocols and requirements of PG asphalt binder have further evolved over the last two decades based on research, test results on supplied asphalt binder and field performance experience. The PG asphalt binder selection in Manitoba is now project specific. Manitoba also adopted split grade (e.g., PG 58-37) grade asphalt binder to make the binder production and supply more convenient and save money from a reduced binder cost, where feasible (e.g., for surface lift). The specified asphalt binder grades also vary based on the depth of AC lifts from the pavement surface to allow for the use of cheaper asphalt binder in lower lifts. The virgin asphalt binder grades must be adjusted to meet the design asphalt binder grades as required for each project when RAP (and Recycled Asphalt Shingles, if approved) are incorporated into the new AC mixes.

Manitoba currently uses the LTPPBind 3.1 software to select the design asphalt binder true grades for each project. The true grades of available virgin asphalt binders should be used to ensure compliance with the true design asphalt binder grade requirements for more cost-effective use of the available virgin asphalt binders. Regardless of the pavement design service life, the PG asphalt binder selection should be based on 20 years accumulative ESALs. The typical asphalt binder grades that are in use in Manitoba are presented in Tables 3.0.11. The supplied asphalt binder must meet the true grade used in the pavement design, as a minimum, corresponding to each specified standard or MSCR grade.

Table 3.0.11: Typical Design Asphalt Binder Grades in Manitoba

Specified Standard Grade	Specified MSCR Grade	True Grade
PG 58-34	PG 58S-34.3	PG 59.6-34.3
PG 58-34P	PG 58H-35.9	PG 64-35.9
PG 58-37P	PG 58H-37.9	PG 64-37.9
PG 64-34P	PG 58V-35.9	PG 70-35.9
PG 64-37P	PG 58V-37.3	PG 70-37.3

The design high temperature grade should be based on pavement temperature at 20 mm below the surface for the top lift (lift 99) and pavement temperature at the top of each underlying lift of AC pavement. The typical high temperature grades for the surface lift at different levels of traffic loads and loading conditions are presented in Table 3.0.12.

Table 3.0.12: Typical High Temperature Grades for Surface Lift at Different Levels of Traffic Loads

Traffic Speed	20 years ESALs		
	<3,000,000	**<10,000,000**	**>10,000,000**
Standard	58S	58H	58H
Slow	58H	58H	58V
Intersection	58H	58V	58E

For all freeways and expressways, and other highways with 20 years design ESALs of 10 million or more, polymer modified asphalt binder should be used in minimum of top two lifts regardless of LTPPBind 3.1 software outcomes. For highways with 20 years design ESALs of 3.0 to 9.9 million, polymer modified asphalt binder should be used in the top lift, as a minimum, regardless of LTPPBind 3.1 software outcomes. The design low temperature grades should be based on the pavement temperature at the surface of each lift. Table 3.0.13 presents the guideline for selecting reliability levels for different types of construction and rehabilitation treatments when determining the design asphalt binder grades. The designer should refer to department's relevant Engineering Standard for any update(s) related to binder selection before recommending the design binder grade.

Table 3.0.13: Selected Reliability for Various Construction and Rehabilitation Treatments

Construction/Rehabilitation Treatments	Selected Reliability Levels	
	High Temp.	Low Temp.
Rubblize PCC and AC Overlay with no GBC interlayer	98%	50%
Rubblize PCC and AC Overlay with ≥100mm thick GBC interlayer	98%	98%
Overlay of Existing AC	98%	50%
Mill and Overlay of Existing AC	98%	50%
Pulverize Asphalt and Overlay	98%	98%
CIR and Overlay	98%	50%
FDR and Overlay	98%	98%
Reconstruction	98%	98%
New Construction	98%	98%

If an AC mix contains RAP, the blend of the virgin asphalt binder and recovered asphalt binder from the RAP should meet the design asphalt binder true grade. The true grades of the recovered asphalt binder from RAP, the proposed virgin asphalt binder and their (i.e., RAP/virgin) blends at 10/90, 15/85, 20/80, 25/75 and 30/70 proportions should be determined in the laboratory. A blending chart should be prepared showing the true grades of the blended asphalt binders at different RAP contents. This blending chart should be used to determine and specify the maximum RAP content and virgin binder grade for each lift of AC pavement for a particular project. The RAP contents should not exceed 15% for the top lift and 25% for other lifts of AC pavement, unless otherwise approved by the Pavement and Materials Engineering Section of the department.

When a blending chart could not be prepared for any valid reason, the RAP binder true grade should be determined in the laboratory and the proposed virgin asphalt binder's true grades should be obtained from the respective asphalt binder supplier(s). The allowable RAP content for a given virgin asphalt binder grade and/or required virgin asphalt binder grades at different RAP contents to meet the design asphalt binder grade requirements can then be determined using the blending formula given by Equation 3.4, which applies to both high and low critical temperatures (FHWA 2011).

$$T_{blend} = T_{virgin} + \frac{RAP_{cont}}{100} * \left(T_{rap} - T_{virgin}\right) \quad (3.4)$$

where,

T_{blend} = critical temperature of blended asphalt binder

T_{virgin} = critical temperature of virgin asphalt binder

PG_{rap} = critical temperature of asphalt binder from RAP

RAP_{cont} = RAP content in percentage

When the true grade of the recovered asphalt binder from RAP is unavailable, Table 3.0.14 can be used as a guideline to select RAP binder grade for preliminary design only.

Table 3.0.14: Estimating Grades of Asphalt Binder Recovered From RAP

Condition RAP	**High Temperature Grade**	**Low Temperature Grade**
Very old and heavily oxidized	Do not use in AC mixes	Do not use in AC mixes
Significantly aged/oxidized	70 (Note 1)	-19
Moderately aged/oxidized	64 (Note 2)	-22
Un-oxidized or slightly oxidized	58 (Note 3)	-25

Note 1: Use 64 for RAP from Provincial Roads in Climate Zones 2 and 3; Note 2: Use 58 for RAP from Provincial Roads in Climate Zones 2 and 3; Note 3: Use 52 for RAP from Provincial Roads in Climate Zones 2 and 3.

The RAP binder grades specified in Table 3.0.14 are applicable to highway sections where the penetration/viscosity grade binders were historically used. For highway sections where PG binders were used, the RAP binder grade should be determined in the laboratory for using Equation 3.3 to determine the virgin asphalt binder grade and allowable RAP contents.

RAP with very old and heavily oxidized asphalt binder should not be used in the asphalt mixes. A RAP will be considered heavily oxidized or very old if the recovered asphalt binder's high and/or low temperature grades are >12 points higher than the true grades of original virgin asphalt binder used in the project from where the RAP is being sourced.

3.8.2 Emulsified Asphalt

An emulsified asphalt is a blend of asphalt binder and water at ambient temperature that contains a small amount of an emulsifying agent (called the surfactant) to hold asphalt globules in suspension. Emulsified asphalts are graded according to their setting time, viscosity, hardness and electrical charge The setting time is the time required for an emulsion to break (colour turns from brown to black) and produce a continuous film of asphalt binder on the aggregate particles on which it is applied. The typical grades are: Rapid Setting (RS); Medium Setting (MS); and Slow Setting (SS). Emulsions are further categorized based on the electrical charge of the asphalt globules. Emulsions with negatively charged globules are called anionic and those with positively charged globules are called cationic.

High Float (HF) emulsions are special types of emulsified asphalt. They are designed with the addition of certain chemicals so that the emulsifier forms a gel structure in the asphalt residue. This gel structure produces a thicker asphalt film on aggregate particles and allows for these emulsions to perform in a wider temperature range than the traditional emulsions with minimal probability of the asphalt draining off during processing and placement. HF emulsions are typically used in chip seals and slurry seals, for stabilization of granular base, RAP and sand, as prime coat on granular base surface and as tack coats on AC or PCC surfaces (TAC 2013, AI 2020).

Manitoba uses SS-1 for tack and prime coats, CSS-1h for cold in-place recycling, CRS-2P and HF-150P for chip seal, HF-500MHR for cold mix and CQS-1hp and CQS-1P for micro-surfacing treatments. RS-1 emulsion can also be used for tack coat.

3.9 Asphalt Concrete (Bituminous) Materials

Asphalt concrete (bituminous) material is a mixture of granular aggregates and asphalt binder. Coarse and fine aggregates of different sizes, grading and quality, and the asphalt binder are mixed in a design proportion to meet the combined aggregate grading as well as different physical and mix volumetric properties requirements. The hot mixed AC are usually produced in a central plant, hauled to the project site and placed on the road with a mechanical paver. Additives are often added to the mix to meet certain performance requirements (such as to address stripping potential), improve viscous properties and allow for the longer haul, late season paving or reduce mixing and compaction temperatures in order to reduce greenhouse gases (e.g., Warm Mix Asphalt). RAP is usually recycled into the new AC mixes for rehabilitation projects that require milling of existing AC pavements or when spare RAP from other projects is stockpiled near the asphalt mixing plant. Each AC mix constituents should be thoroughly mixed at the suppliers specified mixing temperature to form a homogenous mass, placed on the road without causing segregation and compacted (at above the specified minimum temperature) to meet the specified minimum as well as maximum densities and smoothness requirements for ensuring a durable pavement.

Manitoba has been using two AC mixes, named Bituminous Type B (Bit. B) and Bituminous Type C (Bit. C), for new construction, reconstruction and rehabilitation projects. These are dense graded mixtures with finer gradations and lower stiffness than the typical mixes used by other jurisdictions. Manitoba discontinued the use of Bit C mix several years ago due to concern over low stiffness and poor performance. However, Bit. C layer, if identified in an existing pavement, should be reported in the existing pavement investigation report for proper consideration in pavement rehabilitation design, where applicable.

Manitoba has historically used the Marshall Method for the design of AC mixes together with the locally developed specifications for gradation, physical properties and mix volumetric requirements. The Superior Performing (SuperPave or SP) asphalt mix specifications and mix design system, which have been developed under the U.S. Strategic Highway Research Program, have been adopted by most U.S. and Canadian jurisdictions to suit the performance requirements under varying traffic loads and climatic conditions. Accordingly, the SP mix specifications vary depending on the project type with more robust requirements for high, slow and standing traffic loads while less robust requirements for low traffic loads, as opposed to applying a single specification (e.g., Bit. B) for all projects. The mix performance in different climatic conditions is addressed with the selection of appropriate asphalt binder to suit the project specific climatic (high and low pavement temperatures) exposure.

The SuperPave mix design system uses a gyratory compaction technique to produce AC mix specimens, resembling the mix compaction process during AC paving operation, to establish the mix volumetric properties and other requirements as opposed to upright pounding on the specimen surface in the Marshall Method. Manitoba has completed the first SuperPave paving project in 2019 on PTH 1 East. The current specifications of Bit. B and SuperPave mixes are summarized Tables 3.0.15 through 3.0.20.

Table 3.0.15: Combined Aggregates Gradation and Physical Requirements for Bit B. Mix

Passing Sieve Size		**Bituminous Class "B"**	
Metric, mm	***Imperial***	***Lower Limit***	***Upper Limit***
19.0	3/4"	100	100
16.0	5/8"	90	100
12.5	1/2"	75	95
9.5	3/8"	70	90
4.75	#4	55	70
2.00	#10	35	55
0.425	#40	17	32
0.180	#80	4	12
0.075	#200	3	7
Fractured Faces, Min. %		50	
Ironstone Content in Coarse Fraction, Max. %		11 (Top Lift)	
Lightweight Particles Content in Coarse Fraction, Max. %		3 (Top Lift); 7 (Other Lifts)	
L.A. Abrasion Loss, Max. %		35	
Clay Lumps and Friable Particles Content, Max %		1	
Bulk Specific Gravity (Oven Dry Basis) of Coarse and Fine Aggregates, Min. (Note 1)		As specified in the pavement design	
Water Absorption of Coarse and Fine Aggregates, Max. % (Note 1)		As specified in the pavement design	

Note 1: Coarse aggregate refers to material retained on 4.75 mm (#4) sieve; fine aggregate refers to material passing 4.75 mm (#4) sieve and retained on 0.075 mm (#200 sieve). The requirements apply to coarse aggregate and fine aggregate separately.

Table 3.0.16: Bit. B Marshall Mix Requirements

Mix Properties	Bituminous Class B (Bit B)
Voids in Mineral Aggregate (%), minimum	14.0
Voids Filled with Asphalt (%)	67-75
Air Voids Content (%)	4
Effective Asphalt Binder Content (%), minimum	4.5
Marshall Flow, 0.25 mm	8-14
Marshall Stability (kN), minimum	8

Table 3.0.17: Combined Aggregates Gradation and Source Properties for SuperPave Mixes

Passing Sieve Size		SP19		SP12.5		SP9.5		SP4.75	
Metric, mm	Imperial	Lower Limit	Upper Limit	Lower Limit	Upper Limit	Lower Limit	Upper Limit	Lower Limit	Upper Limit
25.0	1”	100	100						
19.0	3/4"	90	100	100	100				
12.5	1/2"	72	90	90	100	100	100	100	100
9.5	3/8"	60	81	76	90	90	100	95	100
4.75	#4	39	62	48	71	57	90	90	100
2.36	#8	23	49	28	58	32	67	55	74
1.18	#16	16	35	19	41	22	48	30	55
0.60	#30	11	25	13	30	14	34	21	39
0.30	#50	7	17	8	21	9	23	14	28
0.15	#100	4	13	4	15	5	17	10	20
0.075	#200	2	8	2	10	2	10	6	13
Ironstone Content in Coarse Fraction, Max. %		11		11		11		11	
Lightweight Particles Content in Coarse Fraction, Max. %		7		3		3		3	
L.A. Abrasion Loss, Max. %		35		35		35		35	
Clay Lumps and Friable Particles Content, Max %		1		1		1		1	
Bulk Specific Gravity (Oven Dry Basis) of Coarse and Fine Aggregates, Min. (Note 1)		As specified in the pavement design		As specified in the pavement design		As specified in the pavement design		As specified in the pavement design	
Water Absorption of Coarse and Fine Aggregates, Max. % (Note 1)		As specified in the pavement design		As specified in the pavement design		As specified in the pavement design		As specified in the pavement design	

Note 1: Coarse aggregate refers to material retained on 4.75 mm (#4) sieve; fine aggregate refers to material passing 4.75 mm (#4) sieve and retained on 0.075 mm (#200 sieve). The requirements apply to coarse aggregate and fine aggregate separately.

Table 3.0.18: Consensus Property Requirements for SuperPave Mixes

Traffic Category (Note 1)	**Combined Aggregate Retained on the 4.75mm Sieve**		**Combined Aggregate Passing the 4.75mm Sieve**	
	Fractured Faces, % Minimum	**Flat and Elongated Particles, % Maximum**	**Uncompacted Void Content of Fine Aggregate, % Minimum**	**Sand Equivalent, % Minimum**
A	55	10	40	40
B	75	10	45 for SP 4.75 and 40 other SP mixes	40
C	85	10	45	45
D	95	10	45	45
E	100	10	45	50

Note 1: Traffic Category is based on the 20 years design traffic loads; "A" = <0.3 million ESALs, "B" = 0.3 to <3.0 million ESALs, "C" = 3.0 to <10.0 million ESALs and "D" = >10.0 million ESALs

The quality requirements of aggregates for AC (both Bit. B and SP) mixes, corresponding to the structural layer coefficient or elastic moduli values used in the pavement design, should be specified in the construction tender and design-build project's technical requirements. Aggregates that exhibit a specific gravity of less than 2.50 and/or water absorption of more than 3.5% can be used in AC mixes if no suitable alternative source is available within a reasonable distance from the project or plant site. In that case, the concerned AC mix should be tested for resilient (elastic) modulus to assign appropriate structural layer coefficient value and adjust the required layer thickness of pavement material(s).

Table 3.0.19: SuperPave Mix Gyratory Compaction Requirements

Design ESALs (million)	**Mix Compaction Parameters**		
	$N_{initial}$	N_{design}	N_{max}
<0.3	6	50	75
0.3 to <10	7	75	115
10 to <30	8	100	160
≥30	9	125	205

Table 3.0.20: SuperPave Mix Requirements

Design ESALs (million)	Required Relative Density, Percent of Theoretical Maximum Specific Gravity			Minimum VMA (%)				Voids Filled with Asphalt (%)	Dust to Binder Ratio
	$N_{initial}$	N_{design}	N_{max}	SP19	SP12.5	SP9.5	SP4.75		
<0.3	≤91.5	96.0	≤98.0	13	14	15	16	70-80[1]	0.6-1.2[4]
0.3 to <3	≤90.5	96.0	≤98.0	13	14	15	16	65-78[2]	0.6-1.2[4]
3 to <30	≤89.0	96.0	≤98.0	13	14	15	16	65-75[2,3]	0.6-1.2[5]
≥30	≤89.0	96.0	≤98.0	13	14	15	16	65-75[2,3]	0.6-1.2[5]

[1] *SP4.75 should have VFA of 67 to 79 percent;* [2] *SP4.75 should have VFA of 66 to 77 percent;* [3] *SP9.5 should have VFA of 73 to 76 percent;* [4] *SP4.75 should have dust to binder ratio of 1.0 to 2.0;* [5] *SP4.75 should have dust to binder ratio of 1.5 to 2.0.*

Tables 3.0.15 through 3.0.20 show that the aggregates and mix requirements vary among the Bit. B (Marshall) and SP mixes. The SP mixes have more stringent requirements than the Bit B. mix. Tables 3.0.18 through 3.0.20 also show that the aggregate source and consensus properties as well as the mix design requirements vary for SP mixes depending on the design traffic loads. These varying requirements for SP aggregates and mixes will provide varying stiffness and other performance parameters of the mixes even though the gradation requirements remain unchanged. In addition, the quality of aggregates can vary among the sources, which can have a significant impact on the mix stiffness and field performance (e.g., moisture damage or stripping) of AC mixes. Therefore, proper characterization of mixes for each variation in gradation as well as aggregate and mix properties are important to ensure appropriate pavement designs for cost-effective highway construction as well as to ensure durable AC pavements. Measures should also be taken to address potential moisture and stripping issues, such as the incorporation of liquid anti-stripping agents or lime into the AC mixes and adequate compaction as well as timely maintenance to prevent moisture infiltration into the AC layer.

The new balanced mix design approach, which is still evolving, attempts to select a mix considering a balance between the fatigue cracking, thermal cracking and rutting performance. Studies are underway to characterize Manitoba's current and proposed mixes for these performance parameters. Any changes to pavement design inputs based on these studies will be reflected in a relevant Engineering Standard of the department.

3.10 Portland Cement Concrete Materials

In a portland cement concrete (PCC) mixture, aggregates (fine and coarse) are bonded together with hardened portland cement paste. Water is added to the mixing process to aid the hydration and bonding process. The added water should be adequate to ensure completion of the hydration process and to aid the placement and finishing of PCC mixes, but not excessive because the excess water causes low strength and poor performance issues. Manitoba uses General Use (GU) cement for the PCC pavements which contains up to 5% limestone. The Cement and Concrete Industry are now promoting the portland General Use Limestone (GUL) cement that contains up to 15% limestone to reduce carbon footprint. Supplementary cementitious materials such as fly ash, slag and silica fume are added to enhance the cement hydration process and durability of PCC.

Regardless of the cement type used, PCC mixes should meet the strength and durability requirements for the traffic loads and project climatic exposure. Flexural strength is the primary PCC mix input in structural design which governs the load carrying capacity of PCC pavements. The durability requirements are resistance to freeze-thaw and de-icing salts, resistance to cracking during the hydration process (due to shrinkage) and over the lifetime, ability to withstand expansion and contraction, and resistance to other distresses including scaling and sulphate attack. The aggregate type also has a significant impact on the expansive properties (coefficient of thermal expansion) of PCC and the resistance to freeze-thaw damage. The aggregates used for PCC mixes should meet the applicable specification requirements to ensure long-term good performance. The balanced mix design approach attempts to optimize the aggregate grading, cementitious and water contents to achieve a balance between strength and durability requirements with a specific focus on reducing the content of the cementitious materials. The PCC mixes should be designed to meet the performance requirements and be characterized to establish the pavement design inputs.

3.11 Aggregate Chip Seal and Micro-Surfacing Materials

Over 20% of Manitoba's highway network consists of aggregate chip sealed surface, locally known as asphalt surface treated (AST) pavements. AST is an application of emulsified asphalt and aggregate chips on gravel or subgrade surface. It is a thin (~20 mm thick) surfacing on roads with low traffic volume to provide a dust free surface, and it is not considered to be a traffic load bearing layer like AC. The current AST surfacing consists of: 1) spraying a thin layer of emulsified asphalt on compacted gravel surface, 2) spreading aggregate chips (small aggregates, typically 9.5 mm in size) by a mechanical spreader, 3) compacting with a roller to ensure bonding between emulsified asphalt and aggregate chips. Since chip seal is subject to spring breakup, double chip seals should be done on gravel base to provide a more durable surface. Fog sealing of chip sealed surface should be considered to avoid stone pick out and enhance the durability of chip seal surfaced pavements.

Chip seal can also be used as a preservation treatment on asphalt paved surface. It retards the progression of asphalt cracking and extend the pavement service life. It also enhances skid resistance and safety (reduced roll over) on roadways.

Micro-surfacing is a preservation treatment to address rutting in asphalt pavements or to provide a new wearing surface with slight improvement in surface smoothness. Micro-surfacing mixtures consists of 100% crushed small (typically 9.5 mm) size aggregates, polymer modified asphalt emulsion, portland cement, water and chemical additives. The 100% crushed stones provide a high resistance to rutting. It also enhances skid resistance and seals transverse and longitudinal cracks on pavement surface.

3.12 Geosynthetic Materials

Several types of geosynthetic materials are commercially available, including different types of geotextile fabrics and geo-grids. Non-woven or woven geotextile fabrics are commonly used in Manitoba for pavement applications. The main purposes of using geotextile fabrics are:

i) To provide added support beneath embankments that are constructed on soft and wet soils or on thin deposits of peat (<1.0 m thick);

ii) To provide a separation between subgrade soils and the granular layer beneath a pavement structure to protect granular layer material(s) from being contaminated by migration of fine, specially silty, soil particles;

iii) To prevent rapid and excessive flow of soil moisture or groundwater into embankments or pavement structures; and

iv) To prevent migration of the fine erodible soils into the voids between the riprap stones used for erosion control.

The added benefit of geotextile fabrics in terms of improved shear strength is negligible for pavement structural design purposes. The geotextile fabrics for the pavement applications other than that are not listed above should be selected based on project and site-specific conditions and requirements. The following applications are a few examples:

i) To support embankments that are constructed on thick deposits of weak compressible soil or peat, greater than 1.0 m thick;

ii) To reinforce earth embankments and retaining walls; and

iii) To provide frost protection, capillary break layers and pavement reinforcement.

Alternatively, geogrids are known to provide considerably increased shear strength and may allow for a reduction in pavement thickness, especially when they are placed between granular material layers or lifts. The benefit associated with the reduction in pavement thickness with the use of geogrids may not be realized in cold climates, like Manitoba, because a reduction in thickness may result in reduced frost protection for typical pavements (total thickness in the range of 400 mm to 900 mm). Further research including local trial application are required to determine the cost effectiveness of geogrid materials for typical pavements in Manitoba.

3.13 Lift Thickness of AC and Granular Materials

When providing the layer thickness requirement of any material, the designer should consider the allowable minimum and maximum lift thickness of that material so that the material can be placed without causing any construction issues. Table 3.0.21 provides the minimum and maximum compacted lift thickness of typical materials.

Table 3.0.21: Minimum and Maximum Compacted Lift Thickness of Typical Materials

Material Type	Min. Thickness (mm)	Max. Thickness (mm)
Bit. B	40	60
SP19	50	70
SP12.5	35	55
SP9.5	25	40
SP4.75	15	25
GBC- I	N/A	125
GBC- II	N/A	100
GBC- M	N/A	100
GBC- S	N/A	100
GSB- C	N/A	150
GSB- F	N/A	225
CR- M50	N/A	200
CR- M100	N/A	350
CR- M125	N/A	400

Chapter 4: TRAFFIC DATA

4.1 Overview

Since roads and highways are constructed for traffic movement, traffic information is a key input for the planning and design of these facilities, including the pavement structures that support the traffic loads. Accordingly, pavement structures in a highway network should be structurally sound to carry the expected traffic loads with a safe and comfortable ride over the design service life. The traffic pattern and volume on a highway or section of a highway may also trigger the pavement reconstruction, rehabilitation, preservation and maintenance options or strategies. Proper collection and accurate estimation of traffic data are therefore important for the design of a reliable pavement structure. For pavement structural design and assessment, the required traffic information is:

i) Traffic volume and vehicle types (configurations);

ii) Percentage or volume of heavy vehicles (truck traffic);

iii) Class distribution of heavy vehicles;

iv) Heavy vehicles growth rate;

v) Directional distribution of heavy vehicles;

vi) Distribution of heavy vehicles among lanes in the design direction;

vii) Axle configurations and number of each axle configuration for each heavy vehicle type; and

viii) Axle weight distributions (weights on various axle types).

The current traffic data such as the annual average daily traffic (AADT), annual average daily truck traffic (AADTT), truck (heavy vehicle) class distribution and the truck traffic growth rate over the design service life should be obtained or calculated from the project Functional Planning Study report, Traffic Impact Study report or Traffic Engineering database. AADT and AADTT data older than five years should not be used for intermediate or final designs but could be used for preliminary design with a projection for the construction year and over the design service life.

The University of Manitoba Urban Mobility and Transportation Informatics Group (UMTIG), in conjunction with the Traffic Engineering Branch of Manitoba Transportation and

Infrastructure, has been compiling all the traffic data collected from the Manitoba highway network. The traffic database and report have been updated annually. This annually updated database and report mainly contain the AADT data for the paved highway network, except for the Automatic Vehicle Classifier (AVC) count stations that also include the truck volume and classification data. However, pavement structural design and assessment require more detailed information on heavy vehicles, including the axle configurations and load distributions, as listed above. To improve the accuracy of pavement structural design and assessment, and for the planned implementation of the Pavement ME Design software, Pavement and Materials Engineering Section of the department developed a database of heavy vehicles and axle load spectra (ALS) in 2008 under a contract with the UMTIG. The database was updated again in 2013 and 2019. The database of heavy vehicles should be updated annually together with the AADT database or separately every five years, as a minimum.

4.2 Traffic Count Stations and Data

As of July 2023, Manitoba has 18 permanent traffic count stations (PCS), 63 Automatic Vehicle Classifiers (AVC) and five Weigh-in-Motion (WIM) stations. A PCS can only count the total number of vehicles passing that location and no weight or classification data is reported. An AVC collects vehicle speed, traffic count, classification and axle spacing data. A WIM collects the axle weight data. In addition, short term traffic counts are taken at approximately 1,500 locations on the highway network, which are called coverage count stations (CCS). Two 48-hour counts are normally conducted at a CCS in a survey year. Traffic survey at these CCS is typically conducted on a three-year cycle. Town counts are also conducted on selected town roads as part of the coverage count program. Two 14-hour (7:00 to 21:00) intersection counts are taken on as-required basis. The intersection counts are two types: i) the FHWA counts that classify vehicles into 15 FHWA vehicle classes, and ii) the car/truck/pedestrian (CTP) counts that classify vehicles into cars, small trucks (FHWA classes 4 to 7), large trucks (FHWA classes 8 to 15) and pedestrians (UMTIG 2020).

4.2.1 Annual Average Daily Traffic (AADT)

The annual average daily traffic (AADT), expressed as the number of vehicles per day (vehicles/day), includes all classes of motorized vehicles from FHWA classes 1 to 15. The vehicle classification scheme used in Manitoba is shown in Figure 4.0.1 (UMTIG 2020). The AADT data available in Traffic Engineering database or annual report are to be used, unless more accurate and recent site-specific data are available from project functional design traffic study or traffic impact study. A short-term traffic volume such as the 14-hour count or the average daily traffic (ADT) should be converted to the AADT using appropriate factors such as the hourly, day of the week and monthly adjustment factors, as applicable. When the applicable adjustment factors are unavailable, the truck traffic volume (i.e., the number of trucks per day) should be estimated in consultation with the involved region, instead of simply estimating from the short-term counts only.

4.2.2 Truck Percentage and Annual Average Daily Truck Traffic (AADTT)

The axle loads on pavements from small vehicles such as motorcycles, passenger cars and pick-up trucks or passenger vans (FHWA Classes 1 to 3) are too small to create any significant impact on pavement structural performance. On average, the impact of 4,000 mid-size cars on flexible pavement or 6,200 cars on rigid pavement is equivalent to the damage caused by one 5-axle truck (Adams & Perry 2018). As such, these smaller vehicles are excluded from the estimate of traffic loads on pavements. The applied axle loads from heavy vehicles are generally used to assess the performance or structural capacity of a pavement and to design pavement structures. The heavy vehicles (or truck) volume, expressed as the annual average daily truck traffic (AADTT), consists of FHWA vehicle Classes 4 to 15 as shown in Figure 4.0.1. However, the axle load spectra in the AASHTOWare Pavement ME Design program consist of vehicle Classes 4 to 13. Accordingly, Manitoba has merged the FHWA vehicle Classes 14 and 15 with the volume of Class 13 to develop a local truck traffic database matching with the Pavement ME Design software requirement.

For pavement design purposes, the current (i.e., the planned construction year) AADTT should be estimated based on the percentage of trucks in the total traffic stream (AADT) on each project site or each section of a highway project.

The AVC stations contain the current AADTT estimates together with the distribution among various vehicle classes. These traffic data are considered as the most reliable (i.e., Level 1) estimates for the highway sections they represent. Truck volume estimates from the percentage of trucks in short-term (14 hours to 48 hours) traffic counts are considered Level 2 data or estimates, and these truck volume estimates are fairly reliable. However, the percentage of trucks from a short-term count should be applied to the AADT (not ADT) for a more reasonable estimate of the AADTT (i.e., truck volume).

<table>
<tr><td></td><td colspan="2">FHWA Class 1 - Motorcycles</td></tr>
<tr><td></td><td colspan="2">FHWA Class 2 - Passenger Vehicles (With 1- or 2-Axle Trailers)</td></tr>
<tr><td></td><td colspan="2">FHWA Class 3 - 2-Axles, 4-Tire Single Units, Pickup or Van (With 1- or 2-Axle Trailers)</td></tr>
<tr><td></td><td colspan="2">FHWA Class 4 - Buses</td></tr>
<tr><td></td><td colspan="2">FHWA Class 5 - 2D - 2 Axles, 6-Tire Single Units (Includes Handicapped-Equipped Bus and Mini School Bus)</td></tr>
<tr><td></td><td colspan="2">FHWA Class 6 - 3 Axles, Single Unit</td></tr>
<tr><td></td><td colspan="2">FHWA Class 7 - 4 or More Axles, Single Unit</td></tr>
<tr><td></td><td colspan="2">FHWA Class 8 - 3 to 4 Axles, Single Trailer</td></tr>
<tr><td></td><td colspan="2">FHWA Class 9 - 5 Axles, Single Trailer</td></tr>
<tr><td></td><td colspan="2">FHWA Class 10 - 6 or More Axles, Single Trailer</td></tr>
<tr><td></td><td colspan="2">FHWA Class 11 - 5 or Less Axles, Multi-Trailers</td></tr>
<tr><td></td><td colspan="2">FHWA Class 12 - 6 Axles, Multi-Trailers</td></tr>
<tr><td></td><td>Manitoba Class 13 - 7 or 8 Axles, Multi-Trailers</td><td rowspan="3">FHWA Class 13</td></tr>
<tr><td></td><td>Manitoba Class 14 - 9 or 10 Axles, Multi-Trailers</td></tr>
<tr><td></td><td>Manitoba Class 15 - 11 or More Axles, Multi-Trailers</td></tr>
</table>

Figure 4.0.1: Manitoba's Standard Vehicle Classification Scheme (UMTIG 2020)

The truck traffic database contains the estimate of AADTT and the percentage of trucks for each section or subsection of the entire paved highway network. The AADTT estimate from an AVC or short-term count station has been transferred/assigned to the adjacent sections on the same highway as deemed appropriate considering no or insignificant drop or bump in the truck volume within that stretch of the respective highway. If no AVC or short-term count is available for a highway section and the AADTT estimate from the nearby count station could not be transferred to that section, the AADTT estimate in the truck traffic database has been provided based on the volume or percentage of trucks in a group of count stations. The grouping (clustering) of the count stations is done based on similarity in activities or travel patterns. The estimated AADTT (truck volume) through this clustering process is considered Level 3 estimate and such estimates are not very dependable for pavement design and analysis purposes.

The AADTT data available in the traffic database should be thoroughly evaluated for reasonableness in consultation with the respective Region, especially for the highway sections with no direct counts of the AADTT or truck percentages. A new truck volume and classification count should be requested if any data appears to be outdated or erroneous. The AADTT data that are Level 3 estimates should not be used in the design without validation with a new count and consultation with the respective Region.

The truck volume on a highway section can increase significantly with attraction due to a change in highway loading classification (e.g., upgrade from B1 to A1 or RTAC and A1 to RTAC), removal of spring weight restrictions, paving of existing gravel road and localized development or other activities. The possible increase in AADTT due to such changes or developments should be estimated in consultation with the respective Region. If no estimate of the increase in AADTT is available, the estimated percentage increase as shown in Table 4.0.1 could be used as a guideline for the design and assessment purposes.

4.2.3 Truck Class Distribution

The distribution (%) of trucks into Manitoba heavy vehicle Classes 4 to 13 should be obtained from the recent classification count or the project's functional design traffic study. If no recent classification count or traffic study is available, the class distribution data available in the traffic database should be used. Special attention should be given to development adjacent to highways and on areas with special activities such as exploration, mining, oil extraction, industrial processing plants and grain elevators for a possible change in the class as well as distribution of axle loads.

Table 4.0.1: Estimated Increase in AADTT Due to Loading Class or Surface Type Upgrade

Current Loading Classification (Surface Type)	Proposed Loading Classification (Surface Type)	Increase in AADTT
A1 (AST or AC)	Seasonal RTAC (AST or AC)	3%
B1 (AST or AC)	Seasonal A1 or RTAC (AST or AC)	5%
A1 (AST, AC or PCC)	RTAC (AST, AC or PCC)	10%
B1 (AST, AC or PCC)	A1 (AST, AC or PCC)	15%
B1 (AST, AC or PCC)	RTAC (AST, AC or PCC)	20%
B1 (Gravel)	B1 (AST, AC or PCC)	25%
Spring Weight Restricted	Non-Spring Weight Restricted	Consult with the respective Region (Note 1)

Note 1: Consider the potential increase in truck volume on the highway in question based on the available non spring weight restricted highways in that area and local activities/demand including potential new developments.

4.2.4 Truck Traffic Directional Distribution

The truck traffic (AADTT) volume or the proportion of total two-way truck volume in each direction of a highway should be taken from the latest traffic study report or traffic database. If the directional distribution of truck volume is unavailable for a highway section, whether undivided or divided, the distribution between two opposite directions should be taken as 50/50.

4.2.5 Truck Traffic on Design Lane (Design Lane Factor)

The proportion of total truck volume on the design lane is called the design lane factor (DLF) or simply the Lane Factor (LF). The design lane is the traffic lane (known as the travel lane in Manitoba) with the highest proportion of total truck volume among all the traffic lanes (in two directions) of an undivided highway, or among all the one-way traffic lanes of a divided highway. Where no project specific data is available, the DLF provided in Table 4.0.2 should be used for calculating the design traffic loads.

Table 4.0.2: Design Lane Factors

Highway Configuration	DLF Based on 2-way Truck Volume	DLF Based on 1-way Truck Volume
Two-lane highways	0.5	1.0
Four-lane urban highways	0.40	0.80
Four-lane rural and semiurban highways	0.45	0.90

When providing a separate pavement design for the passing lane in a special circumstance e.g., to establish a crowned x-section by correcting one-sided slope (known as the sheet drainage) on a highway with two traffic lanes in each direction, the DLF for the passing lane should be taken as 0.30 in urban highways and 0.20 in rural highways based on 1-way truck volume, unless a more accurate data is available.

It should be noted here that the truck axle and gross vehicle weights could be higher in a specific traffic direction than the other traffic direction, depending on the activities and/or supply/demand of goods at the origin versus destination of heavy vehicles. Accordingly, the lane with the highest design traffic loads (determined based on truck volume and applicable truck equivalent factor) will be considered as the design lane of an undivided highway. The required pavement structure for the design lane should be applied to all lanes in both traffic directions of that undivided highway.

For a divided highway, separate pavement design should be provided for each traffic direction. The required pavement structure based on the design lane traffic loads in a traffic direction should be applied to all lanes in that traffic direction.

4.2.6 Axle Configuration and Distribution

The legal axle configurations in Manitoba are single steer axle, tandem steer axle and single, tandem and tridem drive/trailer axles. The single and tandem steer axles are single wheel configurations while the single, tandem and tridem drive and trailer axles are usually dual-wheels configurations. The uptake of the new generation wide base single tires in Manitoba is still low and they are not specifically accounted for in pavement designs.

The type and number of axles per truck vary depending on the truck classification and configuration as shown in Figure 4.0.1. The AASHTO 1993 pavement design guide does not have the axle load equivalency factor (LEF) while the Pavement ME Design program does not have the axle load spectra for steer axles. As such, the single steer axle is treated as a single axle in both design approaches. No guideline is available for the calculation of LEF for tandem steer axle. Since no uptake data of the tandem steer axles is available and the use of tandem steer axles does not appear to be widespread in Manitoba at this time, they are considered as a part of the front steer/single axle count for now for pavement design following the AASHTO 1993 pavement design method. When the data (quantity and weights) for the tandem steer axles is available, each tandem steer axle should be considered as two steer/single axles, each weighing 50% of the gross axle group weight, until more specific LEF or axle load spectra is incorporated into the design approaches. The average axle distribution for each axle and vehicle class based on the currently available data are presented in Table 4.0.3. These data are to be used to develop truck and truck equivalent factors for different classes of trucks and the mixed truck traffic stream until more information is available from different highway classes and activity areas.

Table 4.0.3: Average Axle Distribution for Different Axle and Vehicle Types

Class	Number of Steer/Single Axles per Vehicle	Number of Tandem Axles per Vehicle	Number of Tridem Axles per Vehicle	Total Average Number of Axles per Vehicle
Class 4 (Bus)	1.00	1.09	0.00	3.17
Class 5 (2 Axles ST)	2.00	0.00	0.00	2.00
Class 6 (3 Axles ST)	1.00	1.00	0.00	3.00
Class 7 (4 Axles ST)	0.20	1.60	0.20	4.00
Class 8 (4 Axles TT)	2.17	0.84	0.00	3.85
Class 9 (5 Axles TT)	1.01	1.99	0.00	5.00
Class 10 (6 or 7 Axles TT)	1.00	1.00	1.00	6.00
Class 11 (5 Axles MT)	4.18	0.15	0.17	5.00
Class 12 (6 Axles MT)	3.94	1.02	0.00	6.00
Class 13 (7 or More Axles MT)	1.12	3.33	0.29	8.65

ST = Straight Truck; TT= Tractor-Trailer; MT = Multi-trailers (with Tractor)

4.2.7 Heavy Vehicles Growth Rate

The traffic loads are calculated as the accumulative load repetitions over the design service life or analysis period for pavement structural design and assessment purposes. The growth of truck volume over the design service life or analysis period is required to estimate the accumulative load repetitions. The truck or heavy vehicles compounded annual growth rate should be obtained from the project specific functional study report. If no project specific data of the truck traffic growth rate is available, the truck growth rate can be assumed to be the same as the compounded annual growth rate of AADT (total traffic). The compounded annual growth rate of AADT should be calculated based on the last 10 years of AADT data with a careful assessment of the year-to-year variation. If the calculated growth rate is negative, the growth rate should be taken as zero percent. If no reasonable data of growth rate is available, either for AADT or AADTT, a compounded annual growth rate of 2.0% should be used. The truck growth rates provided in the truck traffic database for the paved network are developed based on limited historical data. These data can be used if they seemed to be reasonable in consultation with the respective Region.

4.2.8 Axle Weight Distribution

The weight data for each axle of each vehicle class is required to calculate the design traffic loads for each highway section. The allowable gross axle and vehicle weights in Manitoba vary depending on the vehicle class (Classes 4 to 13), highway loading classification (B1, A1 and RTAC), type of axles (steer, single, tandem and tridem), axle spread, seasonal increased axle weight allowances in winter (winter weight premiums and winter seasonal A1 or RTAC loads) and seasonal weight restrictions in spring (Level 1 and Level 2 spring weight restrictions). The maximum allowable gross axle and the gross vehicle weights (GVWs) on different classes of highways in Manitoba are presented in Table 4.0.4. The weights listed in Table 4.0.4 are applicable to trucks that meet the minimum inter-axle spacing as well as wheelbase requirements as specified in *Manitoba's Vehicle Weights and Dimensions on Classes of Highways Regulation (MR 155/2018)* and are not equipped with wide base single tires.

During the period of winter seasonal weights, which is specified in the Ministerial Orders during winter season in each year, certain B1 highways are designated as Winter Seasonal A1 highways or RTAC routes and certain A1 highways are designated as Winter Seasonal RTAC routes. During this specified period, a general increase in weight, known as the winter weight premiums, on certain axles are also allowed on all highways/routes. The increased weight allowances are 10% on single axle and 10% on tandem axle up to a maximum of 17,600 kg. There is no increased weight allowance for steer and tridem axles. The gross vehicle weight on a highway or route cannot exceed the legal limit as applicable to the loading classification of that highway or route.

Table 4.0.4: Maximum Allowable Gross Axles and Vehicle Weights (MR 155/2018)

Loading Classification	**B1**	**A1**	**RTAC**	**Super RTAC**
Axle Type	***Weights in Kg***			
Single Steer- Straight Truck	7,300	7,300	7,300	7,300
Single Steer- Truck Tractor with Tandem Drive	6,000	6,000	6,000	6,000
Single Steer- Truck Tractor with Tridem Drive	7,300	7,300	7,300	7,300
Tandem Steer- Straight Truck	11,000	13,600	13,600	13,600
Single Axle	8,200	9,100	9,100	9,100
Tandem Axle	14,500	16,000	17,000	17,000
Tridem Axle (2.4 to <3.0 m axle spread)	20,000	21,000	21,000	21,000
Tridem Axle (3.0 to <3.6 m axle spread)	20,000	23,000	23,000	24,000
Tridem Axle (3.6 to 3.7 m axle spread)	20,000	23,000	24,000	24,000
Tridem Drive Axle (2.4 to <2.7 m axle spread)	20,000	21,000	21,000	21,000
Tridem Drive Axle (2.7 to 2.8 m axle spread)	20,000	21,000	22,000	22,000
Tridem Drive Axle with Tandem Steer (2.7 to 3.1 m axle spread)	20,000	21,000	22,000	22,000
Maximum Gross Vehicle Weights	***47,630***	***56,500***	***62,500***	***63,500***

Alternatively, the allowable axle weights are reduced from the normal (summer/fall) limits on weak and very weak roads during the spring melting period, as specified in the spring road restrictions Ministerial Orders. The allowable axle weights are reduced to 90% of summer weight limits on Level 1 restricted roads (weak roads) and to 65% of summer weight limits on Level 2 restricted roads (very weak roads), with some exceptions for steering axles on Level 1 restricted roads.

Apart from the variation in axle weights due to highway loading classification and seasonal weight allowances or restrictions, the destinations of truck hauls, goods hauled or services mobilized, truck type, axle configurations and localized seasonal activities can affect the axle and vehicle weights. A truck traffic stream also consists of trucks that are fully loaded, partially loaded to varying levels and completely empty. The distribution of axle weights could also be different from the distribution of axle weights in the general mix of trucks (general trips) in special activity areas such as mining, exploration, oil extraction, grain elevators, commercial developments and industrial manufacturing and processing plants. Therefore, comprehensive data for axle weights is required to estimate the traffic loads with a reasonable accuracy for pavement design and assessment purposes.

The weigh-in-motion (WIM) stations, which are installed on Manitoba expressways/primary arterials, provide data from selected locations and they are not likely to be full representatives of the truck traffic variation on highways with different classes, weight levels and regional or local activities. Axle load data from the available WIM stations have been used in this manual to demonstrate the Truck Equivalent Factors, Truck Factors and accumulative ESALs calculation for design and analysis purposes. More extensive axle weight data covering different loading, functional and strategic classes, seasonal variations and regional/local activity types should be collected for a more accurate estimate of design traffic loads. Those data are to be used in the design and analysis of pavements when they are available. Professional judgement should be applied to use the currently available data for each highway section considering the possible variation in axle weights as discussed above.

4.3 Design Traffic Loads

As mentioned in the previous Section, the axle weight distributions in a truck traffic stream consist of different classes of vehicles with varying types as well as the number of axles, and varying axle weights. The Pavement ME Design program uses the actual axle load spectra for each axle type as traffic loads together with the actual number of trucks on the design lane, truck class distribution, the number of each axle type per truck for each vehicle class, monthly variation of truck class and axle weight distributions, and so on. However, in empirical design approaches, like the AASHTO 1993 design guide, a single input of traffic loads is required. Therefore, the total traffic loads over the design service life or analysis period are estimated in terms of the standard single axle load repetitions. Different factors or formulas are used to convert the non-standard single axle loads and loads on other axle types to standard load repetitions for estimating the design traffic loads.

4.3.1 Axle Load Equivalency Factor (LEF)

The standard axle load is referred to an 8,165 kg (18,000 lbs) gross weight on a single axle with dual wheels i.e., a single pass of this standard axle on a pavement is called one standard axle load repetition. The weights on single axles outside this standard load, and all weights on other type of axles, such as steer, tandem and tridem (triple) axles, are converted to the standard load repetitions based on the relative damage to the pavement caused by different axle types and weights as compared to the damage caused by the standard single axle. The standardized load repetitions for a weight on a particular type of axle is called ESAL or the Load Equivalency Factor (LEF) of that axle weight and axle type.

Manitoba has adopted the AASHTO 1993 Guide tables to calculate the LEF for each axle type and weight. In the AASHTO 1993 Guide, the LEF varies depending on the axle configuration (single, tandem and tridem), pavement type (flexible and rigid), pavement strength in terms of structural number (SN) or PCC layer thickness and the desired terminal serviceability at the end of the design service life. To reduce the complexity of LEF calculation for different axle types and axle loads, a terminal serviceability index (Pt) of 2.5 has been selected for both flexible and rigid pavements, a SN of 125 mm (5 in.) has been selected for flexible pavements and a PCC thickness of 250 mm (typical in Manitoba) has been selected for rigid and composite pavements.

In the AASHTO 1993 Guide, steering axle is considered as part of single axle. No LEF table has been provided in AASHTO 1993 guide for quad axles. It should be noted that quad axle is currently illegal in Manitoba, but they can be allowed through special permits. For now, these quad axles have been included in the tridem axle bins for the development of LEFs, but this is subject to change in the future.

At this time, the axle weight data from WIM station (with Quartz sensor) on PTH 190 has been used to determine the LEFs of different axles on both flexible and rigid pavements because of the good accuracy of the data. The LEFs are to be redeveloped once good quality data are available from other WIM stations (after the replacement of existing WIMs that have piezoelectric sensors and installation of few new ones) and axle weights data can be collected from different highways with the variation of loading classes and activities.

4.3.2 Truck Factor (TF)

The total number of standard axle load repetitions due to a single pass of a specific truck type on a pavement section is called the Truck Factor (TF) or ESALs per truck of that truck type. It is the sum of LEFs from all the axles mounted with each specific truck. The TF varies depending on the truck classification and axle combination (number and types of axles), in addition to the weight on each of these axles. The TFs for vehicle Classes 4 to 13 are used to calculate the truck equivalent factor (TEF) for the mixed truck traffic steam at each project location or highway section in Manitoba.

The TFs of different vehicle classes in general traffic stream on flexible and rigid pavements are shown in Tables 4.0.5 and 4.0.6, respectively. These TFs have been developed using the calculated LEFs for varied axle weights on different axle types (i.e., axle load spectra data) of each vehicle class that were recorded at the WIM station on PTH 190. These TFs are considered adequate for Super RTAC routes with a maximum GVW of 63,500 kg.

At the time of developing this manual, no axle load spectra data was available from class A1 and class B1 highways to accurately calculate the AASHTO 1993 TFs for various trucks traveling on these roads. The class B1 and class A1 highways, with less strong pavement structures than that on the RTAC routes, may experience a higher damaging effect (i.e., higher LEF on weaker roads) than that experienced by RTAC routes for a given axle weight. However, the maximum allowable axle and gross vehicle weights are lower on class B1 and class A1 highways than that on RTAC routes. Accordingly, these class B1 and class A1 highways with less strong pavement structures generally experience lighter axle loads than that on RTAC routes. They are also designed for a lower service quality (e.g., lower terminal serviceability index), which corresponds to lower LEFs and TFs. As such, with the application of appropriate vehicle class distribution specific to each highway section, the estimated TEF based on the TFs provided in Tables 4.0.5 and 4.0.6 are considered to be adequate for class B1 and class A1 highways as well. The NHS Core and Intermodal routes within Manitoba (PTH 1, PTH 75, PTH 16, PTH 100, PTH 101 and PTH 190) generally consists of a greater proportion of fully loaded vehicles than other highways. Therefore, the TFs for all classes of vehicles travelling on these highways have been increased by 10%.

Table 4.0.5: TFs for General Mix of Trucks on Flexible Pavement

Highway Loading Class / Heavy Vehicle Classification	All Highways, Except NHS Core and Intermodal Routes	PTH 1, PTH 16, PTH 75, PTH 100, PTH 101 and PTH 190
Class 4 (Bus)	1.317	1.448
Class 5 (2 Axles ST)	0.541	0.596
Class 6 (3 Axles ST)	0.745	0.819
Class 7 (4 Axles ST)	1.549	1.704
Class 8 (4 Axles TT)	0.690	0.759
Class 9 (5 Axles TT)	1.251	1.376
Class 10 (6 or 7 Axles TT)	1.402	1.542
Class 11 (5 Axles MT)	0.827	0.910
Class 12 (6 Axles MT)	1.125	1.237
Class 13 (7 or More Axles MT)	2.516	2.767

ST = Straight Truck; TT= Tractor-Trailer; MT = Multi-trailers (with Tractor).

Table 4.0.6: TFs for General Mix of Trucks on Rigid Pavement

Highway Loading Class / Heavy Vehicle Classification	All Highways, Except NHS Core and Intermodal Routes	PTH 1, PTH 16, PTH 75, PTH 100, PTH 101 and PTH 190
Class 4 (Bus)	1.820	2.002
Class 5 (2 Axles ST)	0.557	0.613
Class 6 (3 Axles ST)	1.078	1.185
Class 7 (4 Axles ST)	2.762	3.038
Class 8 (4 Axles TT)	0.801	0.881
Class 9 (5 Axles TT)	2.033	2.236
Class 10 (6 or 7 Axles TT)	2.685	2.953
Class 11 (5 Axles MT)	1.089	1.198
Class 12 (6 Axles MT)	1.250	1.375
Class 13 (7 or More Axles MT)	4.433	4.877

ST = Straight Truck; TT= Tractor-Trailer; MT = Multi-trailers (with Tractor).

The TFs for flexible pavement should be used for AC, AST and gravel surfaced pavements. The TFs for rigid pavement should be used for PCC and composite pavements. New TFs are to be developed for different highway classes and activity areas when data from all these highways/areas are available. All TFs are to be updated as new data are available (~every five years). The designer should use the updated TFs as specified in the latest version of the department's relevant engineering standard.

In addition to the general mix of trucks, the truck traffic streams on special haul and industry/business access roads usually consist of fully loaded trucks of different classes depending on the type of activities or development such as exploration, extraction and transportation of natural resources (metals, petroleum, aggregates, etc.), industrial manufacturing or processing plants and commercial development along the highways. The TFs presented in Table 4.0.7 should be used for the proportion of the fully loaded trucks in the entire traffic stream on special haul and industry/business access roads with gravel, AST and flexible pavements. The TFs presented in Table 4.0.8 should be used for the proportion of the fully loaded trucks in the entire traffic stream on special haul and industry/business access roads with composite and rigid pavements.

Table 4.0.7: Truck Factors for Fully Loaded Trucks on Flexible Pavement

Loading Class / Heavy Vehicle Classes	B1	A1	RTAC (Including NHS Core and Intermodal Routes)
Class 4 (Bus)	N/A	N/A	N/A
Class 5 (2 Axles ST)	1.688	2.195	2.195
Class 6 (3 Axles ST)	1.516	1.928	2.270
Class 7 (4 Axles ST)	1.391	1.546	1.740
Class 8 (4 Axles TT)	2.191	3.110	3.451
Class 9 (5 Axles TT)	2.020	2.843	3.527
Class 10 (6 or 7 Axles TT)	2.092	2.845	3.446
Class 11 (5 Axles MT)	4.431	6.458	6.458
Class 12 (6 Axles MT)	4.260	6.191	6.533
Class 13 (7 Axles MT)	2.454	4.118	5.144
Class 13 (8 Axles MT)	1.723	3.441	4.838
Class 13 (9 Axles MT)	1.189	2.228	3.337

ST = Straight Truck; TT= Tractor-Trailer; MT = Multi-trailers (with Tractor).

Table 4.0.8: Truck Factors for Fully Loaded Trucks on Rigid Pavement

Loading Class Heavy Vehicle Classes	B1	A1	RTAC (Including NHS Core and Intermodal Routes)
Class 4 (Bus)	N/A	N/A	N/A
Class 5 (2 Axles ST)	1.677	2.254	2.254
Class 6 (3 Axles ST)	2.153	2.923	3.588
Class 7 (4 Axles ST)	2.438	2.828	3.317
Class 8 (4 Axles TT)	2.835	4.181	4.847
Class 9 (5 Axles TT)	3.311	4.850	6.181
Class 10 (6 or 7 Axles TT)	3.931	5.654	7.039
Class 11 (5 Axles MT)	4.444	6.750	6.750
Class 12 (6 Axles MT)	4.920	7.419	8.085
Class 13 (7 Axles MT)	4.033	7.138	9.135
Class 13 (8 Axles MT)	2.743	6.153	9.219
Class 13 (9 Axles MT)	1.992	4.291	6.771

ST = Straight Truck; TT= Tractor-Trailer; MT = Multi-trailers (with Tractor).

4.3.3 Truck Equivalent Factor (TEF)

The distribution (%) of trucks among various classes in a truck traffic stream varies by project location or highway section in Manitoba. The Truck Equivalent Factor (TEF) is the weighted average ESALs per truck of a mixed truck traffic stream i.e., TEF represents the weighted average standard axle load repetitions per truck of a mixed truck traffic stream on a pavement section. It is calculated based on the TF of each truck type (truck class) and the distribution (%) of different truck classes in the mixed truck traffic stream at each project location or highway section. An example of TEF calculation is presented in Table 4.0.9. For a special haul or industry/business access road, the class distribution (%) of both the fully loaded and general freight trucks, their corresponding TFs and their proportions should be used to calculate the combined (weighted average) TEF for the entire truck traffic stream.

Table 4.0.9: An Example of TEF Calculation for Flexible Pavement

Heavy Vehicle Classes	Class Distribution, %	TF (RTAC)
Class 4 (Bus)	0.573	1.317
Class 5 (2 Axles ST)	14.862	0.541
Class 6 (3 Axles ST)	19.025	0.745
Class 7 (4 Axles ST)	2.194	1.549
Class 8 (4 Axles TT)	2.256	0.690
Class 9 (5 Axles TT)	38.894	1.251
Class 10 (6 or 7 Axles TT)	10.328	1.402
Class 11 (5 Axles MT)	0.777	0.827
Class 12 (6 Axles MT)	0.783	1.125
Class 13 (7 or More Axles MT)	10.352	2.516
TEF		***1.186***

4.3.4 Design ESALs

The accumulative standard axle load repetitions on the pavement at each specific project location or highway section over the design service life or analysis period is called the design ESALs. It is calculated based on the total number of trucks per day i.e., AADTT (or AADT and percentage of trucks), DLF, TEF, annual growth and the design service life or analysis period. The following equation (Equation 4.1) can be used to calculate the design life accumulative ESALs for each project or each section of a project:

$$D_{ESALs} = AADTT * DLF * TEF * 365 * \frac{\{(1+GR/100)^N\}-1}{(GR/100)} \quad (4.1)$$

where,

D_{ESALs} = design life (or analysis period) accumulative ESALs
AADTT = annual average daily truck traffic ($= AADT * \%\, Trucks/100$)
AADT = annual average daily traffic
DLF = design lane factor, depending on whether the available AADTT or AADT estimate is for 1-way or 2-way
TEF = truck equivalent factor (ESALs per truck of mixed traffic stream)
GR = annual growth rate (%)
N = design service life or analysis period (years)

Chapter 5: SUBGRADE SOIL STIFFNESS AND DESIGN INPUTS

5.1 Overview

Subgrade soil stiffness, expressed in terms of the resilient modulus (M_R), is a primary input for flexible, semi-flexible and gravel road pavement design using the AASHTO 1993 approach. Resilient modulus is a measure of the elastic response of a soil at a given stress state. It depends on the applied stress, soil confinement, soil type/classification (grain size distribution and plasticity), density and soil composition (moisture content, organic contents, etc.). The resilient modulus can vary seasonally due to the variation in moisture content (due to rainfall, rise/fall in water table, seepage, etc.), subgrade freezing in winter and thawing in the spring.

The resilient modulus of a soil at the desired density (compaction), moisture content, stress and confinement can be determined through the laboratory testing of the representative samples collected from the project site. It can also be estimated based on soil properties and composition or several other measured parameters such as the California Bearing Ratio (CBR) and Dynamic Cone Penetration (DCP) value. However, DCP values can be influenced by the random presence of gravel/stone particles in fine-grained soil layers and it is not suitable for clay with varying gravel contents. As such DCP value is not recommended to use for intermediate and final design, but it can be used for preliminary design purposes if a more accurate measurement of soil stiffness is infeasible. When the measured resilient modulus, CBR or DCP value is unavailable, the Falling Weight Deflectometer (FWD) deflection data from an adjacent highway section can be used to backcalculate the resilient modulus for preliminary design provided that the soil type/classification and composition at the FWD test site closely matches with that of soils at the project site. If no stiffness data is available, the resilient modulus can be estimated from subgrade soil classification (including plasticity) and soil contents (e.g., moisture and organics) for preliminary design purposes.

For rehabilitation and reconstruction projects (without raising the subgrade elevation), the resilient modulus of subgrade soils should be determined through backcalculation with FWD deflection basin data collected from the project sites. If no FWD data is available, alternative approaches as discussed above can be used to estimate the resilient modulus values as applicable for intermediate/final and preliminary designs.

For the rigid (PCC) and composite pavement designs, the AASHTO 1993 design method uses the modulus of subgrade reaction (k) value, which is also called the subgrade support value. It is measured through plate load test on in-situ soils at the project site. In the absence of measured k-Value, it can be estimated from the resilient modulus value.

5.2 Determination of Representative Resilient Modulus

The representative resilient modulus of subgrade soils refers to the resilient modulus value at long term in-situ summer condition of density and moisture content. Such density is typically lower than the density during initial construction of a pavement. Alternatively, the typical long term in-situ summer moisture content of a soil is significantly higher than the optimum moisture content. For example, the typical optimum moisture content of high plastic clay soils found in Winnipeg area is 28 to 29%. The typical long-term summer in-situ moisture content of this soil type is 33 to 35%. As a result, the representative summer resilient modulus value could be 35 to 40% of the measured modulus value at the optimum moisture content.

For a pavement design for new construction, average resilient modulus value of all test results from a uniform section or area can be used as the representative modulus value provided that the coefficient of variation (average divided by standard deviation) of test results does not exceed 10%. Only a few high values (maximum 10% of all data) can be removed as outliers to meet the coefficient of variation requirement when determining the average resilient modulus value. If the coefficient of variation exceeds 10%, the selected subgrade resilient modulus value should be a value with 90% of the test results being above that selected value i.e., only up to 10% test results can fall below the selected representative modulus value. A cumulative distribution of all test data should be plotted and the lowest 10th percentile value from this distribution should be taken as the representative value to meet this later requirement. The same approaches should be used for reconstruction design when the embankment or subgrade will be constructed out of new materials either from within the right-of-way (ROW) of the highway (e.g., material from common excavation) or borrowed from outside the highway ROW.

For reconstruction design without raising the subgrade elevation and for the rehabilitation design, the representative value should be taken as the mean of all backcalculated resilient modulus values, excluding any outliers as discussed in Section 5.2.4.

5.2.1 Laboratory Measured Resilient Modulus

The resilient modulus of unbound materials like the subgrade soil, granular subbase and base depends on the stress state (applied stress and confinement) and the physical properties of materials. It is determined by applying a repeated cyclic axial stress under a static confining stress condition to a cylindrical test specimen. The total resilient (recoverable) axial strain, as the applied load is removed from the specimen, is recorded. The resilient modulus of the material is calculated as the ratio of a given or standard cyclic stress to the corresponding recoverable strain. The resilient modulus test should be conducted following the procedures outlined in the latest version of the *AASHTO T307- Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials*. The density and moisture content of the test specimen should represent the typical in-situ long term summer conditions and the anticipated stress state. The resilient modulus at typical in-situ condition can be determined based on resilient moduli test data at different moisture contents. If the resilient modulus is determined at density and/moisture content that do not represent the typical in-situ long term summer conditions, the measured value should be corrected to represent the typical in-situ long term summer condition. This measured resilient modulus (and adjusted, when applicable) value is the most reliable and thereby the desired option for intermediate and final pavement designs for new construction projects.

5.2.2 Estimated Resilient Modulus from CBR

If the laboratory measured resilient modulus of the subgrade soils from the project site is unavailable, the soaked California Bearing Ratio (CBR) value can be used to estimate the resilient modulus value. The CBR test determines the pressure required for a given penetration of a standard piston into a test specimen. The required pressure for a given penetration (2.54 mm or 5.08 mm) is expressed as a percentage of standard pressure (6.9 MPa for 2.54 mm penetration and 10.3 MPa for 5.08 mm penetration) required for the same penetration depth into a well-graded crushed stone aggregate (which is assumed to have a CBR value of 100).

CBR values are typically smaller at 5.08 mm penetration than that at 2.54 mm penetration for stress softening materials (fine grained soils which exhibit reduced strength or stiffness with increased stress) such as clays and silts. Conversely, the CBR values are typically smaller at 2.54 mm penetration than that at 5.08 mm penetration for stress hardening materials (coarse grained soils which exhibit increased strength or stiffness with increased stress) such as graded gravel or gravelly soils. For pavement design and analysis purposes, the smallest CBR value out of values at 2.54 mm and 5.08 mm should be used.

The CBR test should be conducted following the procedure outlined in the latest version of the *ASTM D1883- Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils* or *AASHTO T193- Standard Method of Test for The California Bearing Ratio*.

The specimens for CBR test should be prepared at typical in-situ density requirement of each soil (as specified in construction specifications, which is usually 95% of the maximum dry density) and optimum moisture content, and soaked following the standard test procedure. The CBR value at this density and soaked condition will be considered to represent the in-situ summer condition. The CBR value of an unsoaked soil at optimum moisture and the specified density will be significantly greater than that exhibited in the field or after soaking, and it should not be used in the design. The CBR value of a soil can also be measured in the field at the project site following *ASTM D4429- Standard Test Method for CBR (California Bearing Ratio) of Soils in Place*. For field CBR measurement, the density and moisture content should represent the actual field conditions.

The resilient modulus value can be estimated from the measured CBR value in soaked condition. The correlation provided in the AASHTO 1993 pavement design guide represents a linear correlation between a static (CBR) test method and a dynamic (resilient modulus) test method, which had a high variability in test data/results. The AASHTO 1993 correlation is also applicable to fine graded subgrade with a CBR value of 10% or less. The data used to develop the above-mentioned correlation had a high variability resulting in the estimated resilient moduli values in the range of 750 to 3,000 times the CBR values. The correlation (Equation 5.1) developed by Rodden et al. (2021) and adopted by the American Concrete Pavement Association (ACPA) to estimate the resilient modulus from a power function of CBR value seems to be more reasonable. The estimated M_R values using this equation are quite similar to estimated M_R values using the equation adopted in the AASHTOWare Pavement ME Design software. The estimated M_R values using Equation 5.1 from the measured CBR values in soaked condition have also shown to match well with the measured or backcalculated resilient moduli values of subgrade soils in Manitoba. As such, Manitoba adopted this correlation to estimate the resilient moduli of native materials for any CBR values in soaked condition.

$$M_R(psi) = 1941.5 * CBR^{0.6845} \qquad (5.1)$$

5.2.3 Estimated Resilient Modulus from DCP

The DCP test is performed in the field to measure the in-situ strength of soils. The principle behind the DCP is that a direct correlation exists between the strength of a soil and its resistance to penetration by solid objects, such as cones. It is a simple test and can be conducted easily with a rugged and inexpensive equipment in different site access conditions. The test is suitable in many soil types including weak rocks. However, the results are highly variable and uncertain for gravelly soils (Newcombe and Birgisson 1999; Christopher et al. 2006).

If a DCP testing is approved by the department due to the unavailability of resilient modulus or CBR data, it should be conducted following *ASTM D 6951- Standard Test Method for Use of the Dynamic Core Penetrometer in Shallow Pavement Applications*. The CBR value of the subgrade soils may be approximately estimated from the DCP value using the following equation (Webster et al. 1992):

$$CBR = \frac{292}{(DCPI)^{1.12}} \tag{5.2}$$

where,

DCPI = DCP penetration index (penetration rate) in mm/blow using an 8.0 kg hammer on a 60° DCP cone

If a 4.6 kg hammer is used, the DCP value should be multiplied by two (2) to calculate the DCPI. The estimated CBR from above equation can be used to estimate the resilient modulus value with a correction factor (say, 0.80) to account for the loss of accuracy due to dual conversions (i.e., DCP to CBR and then CBR to M_R).

5.2.4 Estimated Resilient Modulus from FWD Deflection

For the pavement design purpose, project level FWD data (refer to Chapter 7 for the background and process of FWD data collection) should be collected during the summer-fall months within last three years period of the scheduled construction season. The network level or older project level data can be used for the preliminary designs. The FWD deflection values at each geophone should be corrected to standard stress of 566 kPa (40 kN load applied on a 30 cm diameter FWD load plate) and an effective pavement temperature of 20°C. The resilient modulus should be determined (by backcalculation) for the surface deflection value at each geophone position representing the subgrade (typically, 600 mm to 1800 mm away from the centre of the FWD load plate). The following equation (Equation 5.3) from the AASHTO 1993 design guide should be used to backcalculate the resilient modulus of subgrade at each FWD test point.

$$M_R(psi) = \frac{0.24 * P}{d_r * r} \tag{5.3}$$

where,

M_R = backcalculated resilient modulus (uncorrected), psi

P = applied load, lbs

d_r = measured deflection at radial distance *r* from the centre of the plate (corrected to the standard pavement temperature of 20°C and stress of 566 kPa), inches

r = radial distance from the centre of the FWD load plate at which the deflection is measured (i.e., distance to each geophone position), inches

The representative backcalculated resilient modulus at each FWD test point should be taken for the geophone position that corresponds to a certain minimum radial distance from the centre of the FWD load plate to ensure that the selected M_R represents the stiffness of subgrade at critical depth. Professional judgement should also be applied in the selection of representative geophone location e.g., the selection of a representative geophone location that provides the lowest average M_R from all FWD test points within a highway subsection with a fairly uniform central deflection values in the case of varying radial distances within that subsection. The minimum radial distance should be determined using the following equation from AASHTO 1993 design guide:

$$r \geq 0.7 * a_e \tag{5.4}$$

where,

$$a_e = \sqrt{\left[a^2 + \left(D * \sqrt[3]{\frac{E_p}{M_R}}\right)^2\right]}$$

r = radial distance at which the deflection is measured, inches

a_e = radius of the stress bulb at the subgrade-pavement interface, inches

a = radius of the FWD load plate, inches

D = total thickness of pavement layers above the subgrade, inches

E_p = effective modulus of all pavement layers above the subgrade, psi

M_R = backcalculated resilient modulus (uncorrected i.e., before correction to convert it to a value equivalent to the laboratory measured M_R value), psi

The total thickness (D) can be taken as the average thickness from all cores and boreholes or test pits within each subsection of a project area with fairly uniform central deflection values and applied to each FWD test point within that subsection, especially when the FWD test points do not match with core/bore holes or test pit points. The E_p should be determined using the following equation (Equation 5.5) from the AASHTO 1993 design guide.

$$d_o = 1.5 * p * a * \left\{ \frac{1}{M_R * \sqrt{1 + \left(\frac{D}{a} * \sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1 + \left(\frac{D}{a}\right)^2}}\right]}{E_p} \right\} \tag{5.5}$$

where,

d_0 = deflection measured at the centre of the FWD load plate (corrected to the standard pavement temperature of 20°C and stress of 566 kPa), inches

p = standard stress on the FWD load plate, psi

a = radius of the FWD load plate, inches

D = total thickness of pavement layers above the subgrade, inches

M_R = backcalculated resilient modulus (uncorrected to equivalent laboratory modulus value), psi

E_p = effective modulus of all pavement layers above the subgrade, psi

The representative backcalculated resilient modulus for a highway subsection with uniform strength (fairly uniform central deflection values) should be taken as the average of all representative backcalculated resilient moduli values determined in the earlier step for different FWD test points within that subsection. Any isolated high and unexpected low modulus values (which are considered outliers) should be screened out so that the coefficient of variation (CoV) of a set of backcalculated resilient modulus values, representing a road subsection, do not exceed the limit calculated using Equation 5.6. Isolated area(s) with low modulus (outlier) values should be considered a separate section.

$$CoV = 100 - R \tag{5.6}$$

where,

CoV = coefficient of variation (standard deviation divided by the average), %

R = selected design reliability, %

The representative backcalculated resilient modulus value of subgrade soils on a road section should then be corrected to convert it to an equivalent laboratory measured resilient modulus value. The correction factors (multipliers to the backcalculated resilient modulus values) vary by pavement and subgrade types as well as their stiffness. The recommended values for different pavements with typical subgrade soils are presented in Table 5.0.1.

Table 5.0.1: Correction Factors for Backcalculated Subgrade Resilient Modulus

Existing Pavement Type	Correction Factors
Flexible (AC) and semi-flexible (AST) pavements	0.35
AC over existing rubblized PCC pavements	0.30
Rigid and composite pavements	0.25

5.2.5 Estimated Resilient Modulus Based on Subgrade Soil Types

If no resilient modulus data is available or can be estimated based on any of the methods described earlier, the representative summer resilient modulus can be estimated based on the predominant subgrade soil type and its classification as presented in Table 5.0.2. This estimate is based on typical in-situ density and moisture content with no perceived organics/peats, top-soils and any other deleterious or highly compressive materials. A lower resilient modulus values than that listed in Table 5.0.2 should be selected if the soil moisture content is higher than typical in-situ value for any soil type. A further reduction in resilient modulus will be required if the subgrade soils contain organics and they are not removed from the core of the pavement structures including shoulders.

Table 5.0.2: Estimating Subgrade Resilient Modulus Based on Soil Classification

Soil Type	Unified Soil Classification	AASHTO Soil Classification	Summer Resilient Modulus, MPa
High Plastic Clay	CH	A-7-5	25
Low Plastic or Sandy Clay	CL	A-6/A-7-6	35
Silty/Sandy Clay	CL-ML	A-4	40
Sandy Silt or Silt	ML	A-4	50
Silty Sand or Fine Sand	SM	A-2-4/A-3	60
Granular Fill (GSB-F) (Note 1)	N/A	N/A	90 (Note 2)
Granular Fill (GSB-C) (Note 1)	N/A	N/A	110 (Note 3)
Rock Fill	N/A	N/A	150

Note 1: For materials exhibiting a specific gravity of ≥2.60 and water absorption of ≤2.50%;
Note 2: Use 80 MPa if GSB-F material exhibits a specific gravity of ≥2.50 to <2.60 and/or water absorption of >2.50% to ≤3.50%;
Note 3: Use 100 MPa if GSB-C material exhibits a specific gravity of ≥2.50 to <2.60 and/or water absorption of >2.50% to ≤3.50%.

5.3 Effective Resilient Modulus

The subgrade stiffness varies seasonally due to freezing, thawing and variation in moisture conditions. As a result, the damage to pavement structure varies seasonally with the maximum damage occurring during the spring thaw weakening period. The resilient modulus during the spring could be as low as 20% of the summer/fall value, depending on the soil type and contents. On the other hand, the resilient modulus could be five times or more when the subgrade is frozen in winter. To account for such seasonal variation, the selected design resilient modulus of the subgrade soil should be an annual representative value, which is called the effective resilient modulus, considering the seasonal damage to pavement structure. The effective resilient modulus value should be determined as follows:

Step 1: Calculate the resilient modulus value corresponding to each month of the year using the seasonal factors (multipliers) applied to the summer modulus value as listed in Table 5.0.3. Use smaller factors for spring if subgrade soil contains significant amount (>6.0%) of organics.

In Table 5.0.3, the climate zones are the same as that applicable to Manitoba's spring road restrictions (SRR) and winter seasonal weights (WSW) programs. The climate zone map is shown in Figure 5.0.1. The boundary of each zone is described below:

Zone No. 1A (Southern Manitoba): The Province of Manitoba south and east of Zone No. 1B, and south of the line that includes PR 272 (Duck Bay), going easterly to include PR 513 (Dauphin River) and the northern tip of Black Island, following the eastern shore of Lake Winnipeg to the north shore of the Winnipeg River, easterly along the north shore of the Winnipeg River to PR304, and easterly to the Ontario boundary.

Zone No. 1B (Swan River Area): The Province of Manitoba south of the line that includes PTH 77, going easterly to include PR 483 (Pelican Rapids) and then going southerly to Cowan and south-easterly to Ethelbert to include PTH 10 and PR 367, going southerly to the RMNP boundary, going westerly and then southerly along the RMNP boundary, and westerly to the Saskatchewan boundary to include PR 482 and PR 549 (Shellmouth).

Table 5.0.3: Seasonal Factors for Resilient Modulus Variation Based of Climate Zones

Month	Climate Zones 1A/1B (Southern Manitoba)	Climate Zone 2 (The Pas Area)	Climate Zone 2 (Thompson Area)

January	6.0	6.0	6.0
February	6.0	6.0	6.0
March	3.0	4.0	5.0
*April	0.5	0.5	0.5
*May	0.5	0.5	0.5
June	0.8	0.8	0.8
July	1.0	1.0	1.0
August	1.0	1.0	1.0
September	1.0	1.0	1.0
October	1.0	1.0	1.0
November	1.0	2.0	3.0
December	2.0	3.0	4.0

**Reduce the factors for spring (April/May) to 0.40 if the subgrade/embankment soils contain >6% to 10% organics and to 0.25 if the subgrade/embankment soils contain >10% organics.*

Zone No. 2 (The Pas Area): The Province of Manitoba north of Zone 1A and 1B, and south of the line that includes Sherridon Road (Sherridon), going easterly to include PR 393, Wabowden Access Road (Wabowden) and Sipiwesk Lake Access Road, and easterly to the Ontario boundary.

Zone No. 3 (Thompson and Northern Areas): The Province of Manitoba north of Zone 2.

Step 2: Determine the relative damage in each month of the year using the following equation from the AASHTO 1993 guide:

$$U_f = 1.18 * 10^8 * (M_R)^{-2.32} \tag{5.7}$$

where,

U_f = relative damage in each month

M_R = subgrade soil resilient modulus in each corresponding month, psi

Step 3: Determine the average relative damage (U_{favg}) for the year as sum of the monthly relative damage values divided by 12.

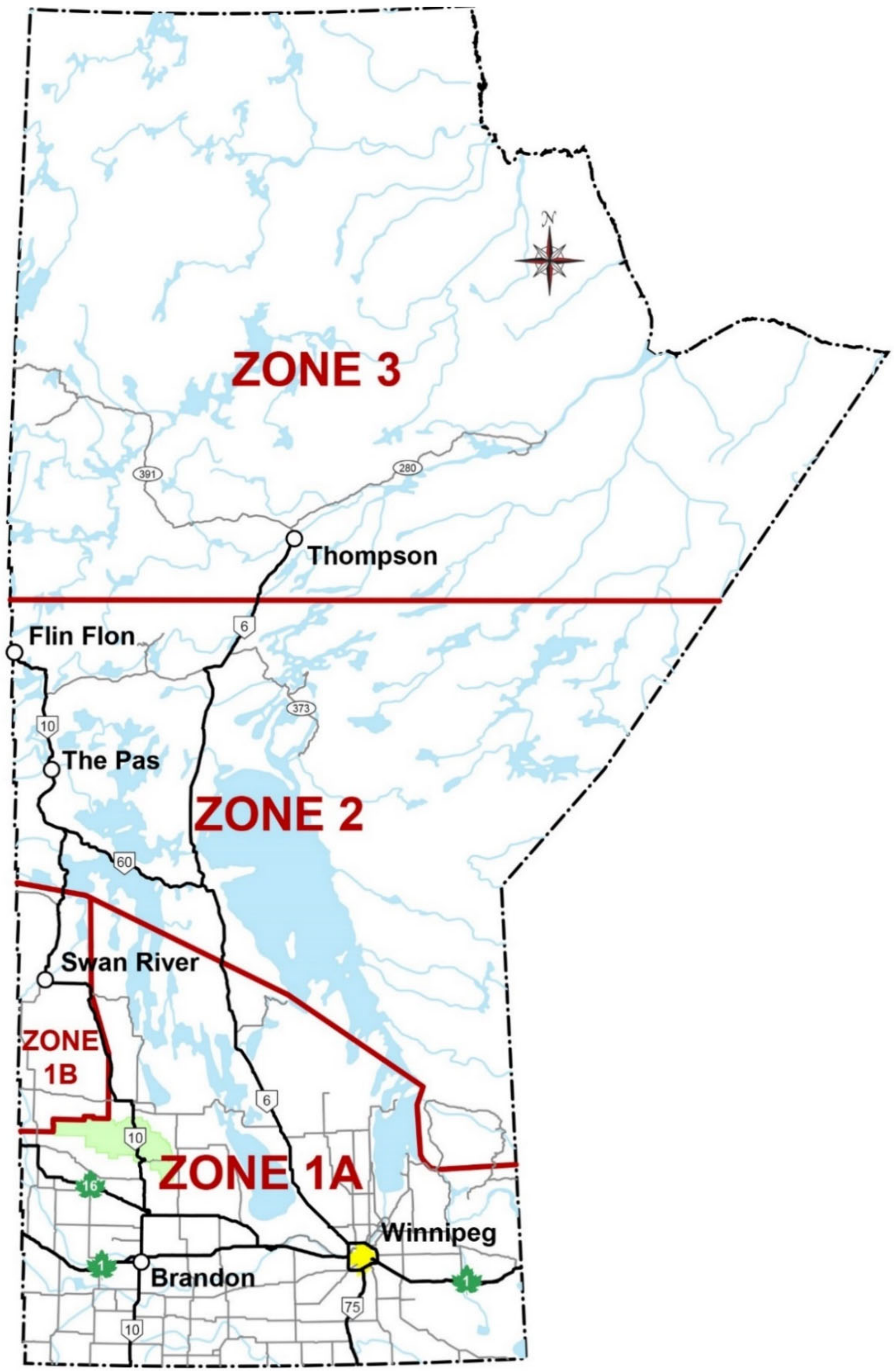

Figure 5.0.1: Manitoba Climate Zone Map

Step 4: Determine the effective resilient modulus of subgrade soils using the following equation (inverse of Equation 5.7) (Christopher et al. 2006):

$$M_R = 3015 * (U_{favg})^{-0.431} \tag{5.8}$$

Step 5: Convert the effective resilient modulus of subgrade soils to metric unit (MPa) by dividing the value in imperial unit (psi) with 145.038, if required.

Example of Effective Resilient Modulus Calculation

Table 5.0.4: Example of Effective Resilient Modulus Calculation

Summer M_R, psi =	**7,098**		
Month	**Monthly Factors**	**Monthly M_R, Psi**	**Monthly Rel. Damage**
January	6.0	42,591	0.00215
February	6.0	42,591	0.00215
March	3.0	21,295	0.01072
April	0.5	3,549	0.68481
May	0.5	3,549	0.68481
June	0.8	5,679	0.23015
July	1.0	7,098	0.13715
August	1.0	7,098	0.13715
September	1.0	7,098	0.13715
October	1.0	7,098	0.13715
November	1.0	7,098	0.13715
December	2.0	14,197	0.02747
Sum of Relative Damage Values			2.32798
Average Relative Damage			0.19400
Effective M_R, psi			**6,113**
Effective M_R, MPa			42.1

5.4 Effective Modulus of Subgrade Reaction

The modulus of subgrade reaction (k), also called the Westergaard modulus of subgrade reaction, value is one of the primary inputs for the design of rigid pavement structures. The k-Value is a quantitative estimate of the degree of support provided by pavement foundation (subgrade) and subbase/base layer(s) underneath a portland cement concrete (PCC) surface layer. The k-Value can be determined by a non-repetitive plate load test at the project site following *AASHTO T222 (Standard Method of Test for Non-repetitive Static Plate Load Test of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements)* or *ASTM D1196 (Standard Test Method for Non-repetitive Static Plate Load Tests of Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements)*.

There is no direct laboratory procedure for determining the k-Value of pavement foundation. When time and equipment are not available to determine site-specific k-Value for the subgrade soil, it should be estimated using correlation with soil strength/stiffness parameters. Subbase/base layer(s), placed over the subgrade, provide an increase in support value. Therefore, k-Value should be adjusted to account for the increased support value from subbase and base layers when determining the PCC slab thickness.

Manitoba has been using 200 mm of subbase and 100 mm of base below the PCC surface since 1990's for jointed and doweled PCC pavements. However, those PCC pavements, placed over the locally encountered soft/weak (typically) high plastic clay subgrade soils, have shown to experience poor long-term performance, especially in terms of highly degrading surface smoothness. Design trials using the AASHTOWare Pavement ME Design software also indicated that local fine-grained soils (AASHTO classification A-7 to A-4) subgrade and cold climate with a high freezing index are poor combination when comes to PCC pavement design and performance. No design meets the desired performance criteria regardless of the thickness of PCC and/or base layer(s) thickness under the condition specified above. However, a layer of granular subgrade helps dramatically to meet the performance targets. As such, all new PCC pavement design and construction should include an additional granular layer below the base layer if the native subgrade or borrowed embankment materials are graded as A-4, A-5, A-6, A-7-5 and A-7-6. When using the AASHTO 1993 Design approach, this granular layer on fine-grained soils should be called subbase to determine the composite k-Value. In the AASHTOWare Pavement ME Design approach, this granular layer should be considered as granular subgrade.

The AASHTO 1993 design guide recommends estimating k-Value from the correlation with subgrade resilient modulus value for new construction and reconstruction designs. The estimated k-Value is required to be adjusted for subbase and base layers stiffness and thickness to calculate the effective composite k-Value for use as design input considering the seasonal variation of moduli of all supporting layers including the subgrade. For rehabilitation designs, the AASHTO 1993 guide recommends determining the composite k-Value through backcalculation from FWD deflection test data. Manitoba estimates the subgrade k-Value from the backcalculated subgrade resilient modulus and then determines the composite k-Value using the same approach as used in designs for new construction or reconstruction.

The AASHTO 1993 design method also recommends correcting the composite k-Value for potential loss of support due to the erosion of granular material and limits the granular layer seasonal modulus to four times the seasonal modulus of the subgrade. Manitoba never considered these factors or aspects in PCC pavement design. ACPA/American Concrete Institute (ACI) design methods such as StreetPave and PavementDesigner also do not consider them in the PCC pavement designs.

A correction to reduce k-Value for loss of support deemed unnecessary for typical subbase/base materials and construction practices used in Manitoba. Rather, subbase/base materials should be selected and construction practices should be adjusted to avoid erosion of these materials and loss of support. As such, no correction is required for loss of support when using typical unbound subbase/base, stabilized subgrade, or cement/asphalt treated base/aggregate materials. If any such erosion, which results in voids underneath the PCC slab, is experienced in any in-service PCC pavement slab, measures should be taken to fill the voids, and to raise the slab to the desired elevation, where required.

By limiting the seasonal moduli values of subbase material to four times the seasonal resilient moduli values of subgrade, agencies cannot take the advantage of good quality subbase material because reduced subbase seasonal moduli values will result in a reduction in the effective k-Value and increase in the required PCC slab thickness. In practice, stiffer subbase materials are shown to provide stronger support for all pavements. Any plunging of a stiffer subbase layer into a weak subgrade is expected to be uniform throughout a project section. In addition, a thicker jointed PCC pavement does not necessarily mean an improved performance, especially when the subgrade/foundation support is weak. As such, there is no need to limit the subbase moduli to four times the modulus of subgrade to increase PCC slab thickness. Rather, consideration should be given to treat subgrade material with cement or lime for a stronger support and to reduce the required subbase thickness, if practically and economically feasible.

In the AASHTO 1993 pavement design approach, the effective composite k-Value is also dependent on the PCC slab thickness and a shallow depth to underlying rock layer. However, the effect of varied PCC slab thickness on the effective composite k-Value was found be very small when using seasonal modulus of subgrade and subbase/base layers. There is no effect of varied PCC slab thickness on the calculated composite k-Value when using the effective modulus of subgrade and annual equivalent moduli of subbase/base layers. The effect of varied PCC slab thickness was never a consideration for PCC pavement design in Manitoba. It is not considered in the ACPA/ACI design methods as well. Therefore, the effect of varied PCC slab thickness on the composite effective k-value is ignored in this design manual. The rock layer is well below the design subgrade elevation on highways where PCC pavements are typically constructed. Therefore, the effect of rock layer is also excluded from effective composite k-Value determination.

Based on the assessment presented above, Manitoba developed a new approach for determining the effective composite k-Value for reasonable design thickness of PCC pavements. The following steps can be used for determining the effective composite modulus of subgrade reaction for jointed PCC pavement design:

Step 1: Identify the subgrade and subbase/base type(s), thicknesses, physical properties and soil contents.

Step 2: Determine or estimate the subgrade M_R value at summer in-situ condition. Use appropriate correction factor to convert the backcalculated M_R value from FWD deflection data to equivalent laboratory measured M_R value, where applies. Determine the effective resilient modulus of subgrade using the procedure outlined in Section 5.3.

Step 3: Determine or estimate the resilient modulus value of unbound subbase and base at summer in-situ condition (refer to Chapter 6). Determine the equivalent annual resilient (elastic) moduli of subbase and base materials using the procedure outlined in Section 6.10.3 (Chapter 6). The equivalent annual moduli of typical base and subbase materials are presented in Table 6.0.10 (Chapter 6)

Step 4: Use the following equation (Equation 5.9) to determine the effective composite k-Value (AASHTO 1986: Volume 2 and Christopher et. al. 2006) of subgrade and subbase layers:

$$\ln (k_c) = -2.807 + 0.1253 (\ln D_{SB})^2 + 1.062 (\ln M_R) + 0.1282 (\ln D_{SB}) (\ln E_{SB}) - 0.4114 (\ln D_{SB}) - 0.0581 (\ln E_{SB}) - 0.1317 (\ln D_{SB}) (\ln M_R) \quad (5.9)$$

where,

k_c = effective composite modulus of subgrade and subbase (pci)
D_{SB} = thickness of subbase material (inches)

E_{SB} = elastic modulus of subbase material (psi)
M_R = resilient modulus of subgrade (psi)

Step 5: Convert the effective composite k-Value of subgrade and subbase layer into effective composite modulus (M_{Rc}) of subgrade and subbase layer using the following equation (AASHTO 1993, Rodden et al. 2021):

$$M_{Rc} = 19.4 * k_c \qquad (5.10)$$

Step 6: Use the following equation (Equation 5.11, which is the same as Equation 5.9) to determine the effective composite k-Value of the foundation support (subgrade, subbase and base layers combined):

$$\ln (k_{c\text{-}f}) = -2.807 + 0.1253 (\ln D_B)^2 + 1.062 (\ln M_{Rc}) + 0.1282 (\ln D_B) (\ln E_B) - 0.4114 (\ln D_B) - 0.0581 (\ln E_B) - 0.1317 (\ln D_B) (\ln M_R) \qquad (5.11)$$

where,

$k_{c\text{-}f}$ = effective composite k-Value of the foundation support (pci)
D_B = thickness of base material (inches)
E_B = elastic modulus of base material (psi)
M_{Rc} = effective composite modulus of subgrade and subbase layer (psi)

Step 7: Convert the corrected $k_{c\text{-}f}$ to metric unit (kPa/mm), if required, by multiplying the value in imperial unit (pci) by 6.894757 and then dividing the product by 25.4.

Example Calculation of Effective Foundation Support (k-Value)

Table 5.0.5: Example of Effective Composite Subgrade Support (k-Value) Calculation

Subbase Material =	**CR-M50**		Subbase Thickness, inches =		**12**
Base Material =	**GBC I**		Base Thickness, inches =		**8**
Note: Limit of Subbase and Base Resilient Modulus = 15,000 to 45, 000 psi					
			Subgrade and Subbase Composite k-Value		
Materials	Effective Modulus of Subgrade, psi	Annual Equivalent Modulus of Subbase, psi	LN Composite k-Value, pci	Composite k-Value, pci	Composite k-Value, KPa/mm
Subgrade and Subbase	3,122	33,510	5.57	262.47	**71.25**
			Subgrade, Subbase and Base Composite k-Value		
Materials	Subgrade and Subbase Composite Modulus, psi	Annual Equivalent Modulus of Base, psi	LN Composite k-Value, pci	Composite k-Value, pci	Composite k-Value, KPa/mm
Subgrade, Subbase and Base	5,092	32,490	5.77	321.27	**87.21**

The above-described approach is applicable only to untreated subgrade, subbase and base materials below a PCC layer. If a cement, asphalt or lime stabilized subgrade, cement or asphalt treated subbase or base or a lean concrete base layer is placed, the ACPA Subgrade k-Value Calculator available at http://www.apps.acpa.org/apps/kvalue.aspx should be used to determine the composite k-Value using the effective modulus of each support layer. A maximum three subbase/base layers can be entered into the ACPA tool for composite k-Value calculation. Combine all adjoining bound (stabilized/treated) layers into one layer and all adjoining unbound layers into another layer, if required. The thickness of any layer material should not exceed 450 mm (18 in.) and the thickness of any unbound material (granular base and subbase) layer should not be less than 100 mm (4 in.) when calculating the effective composite k-Value.

If the total thickness of base/subbase/granular fill layer(s) below a PCC layer is 1.0 m or greater, it is likely to act as a subgrade foundation. As such, the total thickness of base and subbase materials including any granular fill material(s) should be limited to a maximum of 900 mm (36 in.) when determining the effective composite k-Value. If the total thickness of base/subbase/granular fill layer(s) below a PCC layer is 1.0 m or greater, the effective composite k-Value should be estimated using the modulus of the weakest material, considering it a subgrade placed directly below the PCC layer.

Chapter 6: DESIGN OF FLEXIBLE AND SEMI-FLEXIBLE PAVEMENTS FOR NEW CONSTRUCTIION AND RECONSTRUCTION

6.1 Design Inputs

The input parameters for pavement design using the AASHTO 1993 Design Guide approach for new construction and full depth reconstruction projects are:

i) Design life and ESALs
ii) Subgrade stiffness
iii) Subgrade soils frost heave potential
iv) Pavement serviceability
v) Design reliability
vi) Overall standard deviation
vii) Drainage and environmental conditions
viii) Pavement layer material properties

6.2 Design Life and ESALs

For new construction and reconstruction projects, AC and AST pavements should be designed to provide 20 years initial service life at a preselected minimum service quality without any structural enhancement or AC resurfacing. A shorter design life can be selected in special cases, e.g., for passing lanes on two-lane highways, turning lanes and cut-off lanes where the adjacent existing lane(s) will be rehabilitated within the next 10 years, temporary roads, temporary crossovers and detours, and locations with frost susceptible subgrade soils. The design traffic loads i.e., the accumulative standard road repetitions or ESALs over the selected design service life should be calculated using Equation 4.1 with the appropriate TEF as outlined in Chapter 4. All routes classified as trade or commerce in department's strategic classification system should be designed to handle RTAC loads regardless of traffic volume and functional classification.

6.3 Subgrade Soil Stiffness

For economic pavement structures, the project length on a particular highway section can be subdivided into smaller subsections based on the uniformity in subgrade stiffness values and ease or effectiveness of construction activities. Generally, a subsection length should not be less than 2.0 km, unless the total length of the highway section under construction is less than

2.0 km in length. Once the highway section is divided into subsection(s), the representative and the effective resilient modulus for each subsection should be determined following the procedures described in Chapter 5.

6.4 Subgrade Soils Frost Heave Potential

Subgrade soil frost heave in winter and settlement during spring thawing seasons cause pavement deterioration and loss of serviceability. As indicated earlier, three conditions must be present for frost heave to occur: i) presence of frost susceptible soils, ii) presence of moisture and iii) freezing weather. As such, information of project site related to frost heave issues should be collected before considering frost mitigation measures including any increase in granular subbase/base layer(s) thickness. The following information should be collected from local maintenance staff:

i) Severity of frost heave and settlement i.e., how bad is the frost heave and settlement issues, in the concerned area of a highway section or subsection. Table 6.0.1 provides guideline for assessing the frost heave and settlement severity levels.

Table 6.0.1: Guideline for Frost Heave Severity Classification

Classification	Definition
Very severe	Very high frost heave and settlement issues causing extreme concern and most frequent complaints
Severe	High frost heave and settlement issues causing major concern and frequent complaints
Medium	Moderate frost heave and settlement issues causing significant concern and occasional complaints
Low	Noticeable frost heave and settlement issues causing some concern and few complaints
Negligible or very Low	No noticeable or minor frost heave and settlement issues causing no considerable concern and no or rare complaints

ii) Frost heave interval i.e., average spacing (distance) between successive frost heave locations; e.g., >250 m, 150-250 m, 100-150 m, 75-100 m, 50-75 m, 30-50 m, 20-30 m, 10-20 m and <10 m.

iii) Frequency of occurrence i.e., repetition of frost heave occurrence on an annual basis; e.g., always (~ every year), frequent (~every 2-3 years), sometimes (~every 4-5 years), occasional (~every 6-7 years) and rare (>every 7 years) or never.

Once the above information is collected, the pavement serviceability loss due to frost heave should be calculated to determine the serviceability loss due to traffic and the corresponding pavement structure requirements.

6.5 Design Serviceability Loss Due to Frost Heave

The design environmental serviceability loss due to frost heave should be estimated based on potential maximum serviceability loss due to frost, frost heave probability, frost heave rate and time (service life). AASHTO 1993 Pavement Design Guide provided a chart for estimating the design environmental serviceability loss due to frost heave. For pavement design and analysis purposes, the following equation (Equation 6.1), which is provided in the AASHTO 1993 Guide- Appendix G, can be used to estimate the serviceability loss due to frost heave.

$$\Delta PSI_{FH} = 0.01 * DPSI_{MAX} * P_F * [1 - e^{-(0.02*j*t)}] \tag{6.1}$$

where,

ΔPSI_{FH} = design environmental serviceability loss due to frost heave

ΔPSI_{MAX} = potential maximum serviceability loss due to frost heave

P_F = frost heave probability

φ = frost heave rate, mm/day

t = design (service) life or performance period

Potential Maximum Serviceability Loss Due to Frost Heave

The potential maximum serviceability loss due to frost heave is a function of subgrade materials drainage quality and frost depth. The subgrade soils in Manitoba are generally fine graded materials with poor drainage quality. The depth of frost penetration into the subgrade varies with location. The guideline presented in Table 6.0.2 has been developed for determining the frost penetration into subgrade based on observed frost depths in different climate zones (Figure 5.0.1) in Manitoba and correlation with the cumulative freezing indices. The potential maximum serviceability loss due to frost heave can be estimated from graph provided in the

AASHTO 1993 Pavement Design Guide, Figure G.7 "*Graph for Estimating Maximum Serviceability Loss Due to Frost Heave*" knowing the frost penetration depth and subgrade drainage quality. Equation 6.2 has been developed (based on the graph provided in the AASHTO 1993 Pavement Design Guide) for estimating the potential maximum serviceability loss due to frost heave for Manitoba subgrade soils that has poor drainage quality.

Table 6.0.2: Maximum Frost Depth and Frost Penetration into the Subgrade

Climate Zone	Total Frost Depth	Frost Penetration into Subgrade
Zones 1A and 1B (South of The Pas)	2.0 m	Total frost depth minus the estimated thickness of pavement structure
Zone 2 (The Pas Area)	2.2 m	
Zone 3 (Thompson Area)	2.5 m	

$$DPSI_{MAX} = 1.3128 * FP_{SG} \quad (6.2)$$

where,

FP_{SG} = frost penetration depth into the subgrade

Frost Heave Probability

As per AASHTO 1993 Pavement Design Guide, frost heave probability depends on several factors that include: a) the extent of frost susceptible soils, b) moisture availability, c) pavement drainage quality, d) number of freeze-thaw cycles in a year, and e) the depth of frost penetration. However, there is no clear-cut method to estimate the frost heave probability and designers should rely heavily on local experience. Therefore, Manitoba has developed its own guideline to estimate the frost heave probability as a function of locally observed extent of frost heave and its time frequency of occurrence. These two factors take into account all the factors stated in AASHTO 1993 Pavement Design Guide. The frost heave probability can be calculated using Equation 6.3.

$$P_F = Ext_{FH} * Freq_{FH} \quad (6.3)$$

where,

P_F = frost heave probability in percentage

Ext_{FH} = extent of frost heaving

$Freq_{FH}$ = time frequency factor for frost heave occurrence

The extent of frost heave should be estimated based on the average spacing (distance interval) between successive frost heaves along a road section that experiences frost heaving and settlement issues. The frost heave extents based on Manitoba's local conditions are presented in Table 6.0.3.

Table 6.0.3: Estimation of the Frost Heave Extent

Frost Heave Interval, m	**>250**	**150-250**	**100-150**	**75-100**	**50-75**	**30-50**	**20-30**	**10-20**	**<10**
Ext_{FH}, %	0*	30	40	50	60	70	80	90	100

**Only localized measures should be considered if the frost heave interval is >250 m and frost heaves are severe or very severe.*

The time frequency factor for frost heave occurrence is a function of time interval between successive annual frost heaving experiences within the functional or design life of a pavement. The time frequency can be estimated from the guideline presented in Table 6.0.4.

Table 6.0.4: Estimation of Time Frequency Factor for Frost Heave Occurrence

Frequency	**Always**	**Frequent**	**Sometimes**	**Occasional**	**Rare or Never**
Time Interval	(~Every year)	(~Every 2-3 years)	(~Every 4-5 years)	(~Every 6-7 years)	(>7 every years)
Frequency Factor	1	0.75	0.5	0.25	0

Using Tables 6.0.3 and 6.0.4, when the frost heave interval is 50-75 m and the frequency of repeated occurrence is every 2-3 years, the frost heave probability is 0.60*0.75 = 45%.

Frost Heave Rate

Frost heave rate refers to the rate of increase in road roughness, in millimetres per day, which is associated with frost heaving of subgrade soils. The frost heave rate depends on soil classification, percentage of particles by weight smaller than 0.02 mm, subgrade soils frost group, soil frost severity classification and plasticity index as shown in Figure 5.0.1. Estimating the frost heave rate using this chart is somewhat subjective and complex due to overlapping blocks of each frost group, frost heave rate and soil classes. To ease or simplify the design and analysis process for frost susceptible soils, Manitoba has developed a table for the selection of reasonable frost heave rate for various soil types.

The selected frost heave rates for different soils are shown in Table 6.0.5. As mentioned in a previous chapter, gravelly and sandy soils containing less than 6% particles by weight smaller than 0.02 mm, which are considered suitable as subbase materials, are excluded from frost heave consideration in pavement design and analysis. The guideline for frost severity classification is presented in Table 6.0.1. This frost susceptibility classification was selected based on field observation of the severity of frost heave and settlement due to frost melting. A very low frost heave issue, with a frost heave rate of 1.0 mm/day or less, is expected to be eliminated with the application of pavement preservation and/or rehabilitation treatments (e.g., overlay or mill and overlay), and therefore, they are ignored in pavement design and analysis.

Performance Period

Performance period is the service life of a pavement structure considering serviceability loss due to traffic loads and environmental serviceability loss due to the frost heave.

6.6 Pavement Serviceability

The pavement serviceability is a measure of pavement performance that comprises surface smoothness or roughness (irregularities), wheel path rut depth and degree of cracking. The AASHTO 1993 Pavement Design Guide refers the pavement serviceability levels in terms of Present Serviceability Index (PSI). It is rated based on a 5-point scale where five indicates a perfect road and 0 (zero) indicates an impassable road. From a ride perspective, as established by the AASHTO Road Test Expert Panel, a rating of 2.5 was considered unacceptable to 55% and a rating of 2.0 was considered unacceptable to 85% of raters.

The PSI value after construction is termed as the Initial Serviceability Index (p_0) while the PSI value at the end of service life is called the Terminal (or failure) Serviceability Index (p_t). The p_0 values depend on the quality of construction and while the p_t values are selected by each agency based on local conditions and needs (e.g., based on desired serviceability, safety and financial prudence). Typically, a lower p_t value is used for low volume and/or secondary highways/roads.

Table 6.0.5: Selected Frost Heave Rates for Different Subgrade Soils in Manitoba (after AASHTO 1993, U.S. Army 1984)

Frost Group	Unified Soils Classification	% Finer Than 0.02 mm by Weight	Frost Susceptibility Classification	Average Frost Heave Rate, mm/day
F1/F2	GW-GM, GP-GM and GM	6 – 20	Low	1.5
			Medium	3.0
			High (Severe)	4.5
F2	SW-SM, SP-SM, SM	6 – 15	Low	1.5
			Medium	3.0
			High (Severe)	6.0
			Very High (Very Severe)	9.0
F3	GM, GC, GM-GC	>20	Low	1.8
			Medium	3.0
			High (Severe)	4.5
	SM-SC, SC	>15	Low	1.8
			Medium	3.0
			High	5.5
	CL, CH (PI ≥ 12)	-	Low	1.5
			Medium	3.0
			High (Severe)	6.0
			Very High (Very Severe)	9.0
F4	ML, MH	-	Low	2.0
			Medium	4.0
			High (Severe)	8.0
		<30	Very High (Very Severe)	15.0
		>30	Very High (Very Severe)	20.0
	SM	>15	Low	1.8
			Medium	3.0
			High (Severe)	6.0
	CL, CL-ML (PI<12)	-	Low	1.5
			Medium	3.0
			High (Severe)	6.0
			Very High (Very Severe)	10.0

Manitoba has been using 4.5 as the p_0 value for all highways/roads regardless of quality of construction and initial pavement surface smoothness. The selected p_t value was 2.5 regardless of highway classification and traffic volume. A change was required to reflect the quality of construction and ride that are actually being currently achieved after the construction when selecting the p_0 values. For example, an asphalt surface treated (AST) pavement or thin AC pavement cannot be constructed as smooth as a thick pavement with multiple asphalt concrete lifts. Based on the post-construction relative smoothness data on Manitoba highway construction projects, a new guideline for p_0 values has been developed.

A decision was also made to use lower p_t values for low volume and secondary highways for better management of allocated budgets from a pavement management perspective. This strategy is expected to improve the network health as savings from secondary or low volume highways can be invested to other primary and high traffic volume highways. The p_t values are now dependent on the highway classification and total traffic volume. Tables 6.0.6 and 6.0.7 present the guidelines for the selection of p_0 and p_t values, respectively.

Table 6.0.6: Guideline for Initial Serviceability Index (p_0)

Surface Layer	**Initial PSI (p_0)**
AST	4.0
1 lift AC	4.1
2 lifts AC	4.2
3 lifts AC	4.3
4 lifts AC	4.4
>4 lifts AC	4.5

Table 6.0.7: Guideline for Terminal Serviceability Index (p_t)

Highway Classification	**AADT**	**Terminal PSI (p_t)**
Freeway, Expressway and Primary Arterial	N/A	2.5
Secondary Arterial and Trade/Commerce Routes other than Freeway, Expressway and Primary Arterial	N/A	2.4
Collector, Service and Access Roads	>2,000	2.3
Collector, Service and Access Roads	750-2,000	2.2
Collector, Service and Access Roads	250-750	2.1
Collector, Service and Access Roads	<250	2.0

AASHTO 1993 design method requires design serviceability loss due to the traffic load repetitions as the design input. The design serviceability loss due to traffic loads should be calculated as follows:

$$\Delta PSI_{TL} = p_0 - p_t - \Delta PSI_{FH} \tag{6.4}$$

where,

ΔPSI_{TL} = serviceability loss due to the total traffic loads over the design service life

ΔPSI_{FH} = serviceability loss due to frost heave over the design service life

6.7 Design Reliability

The design reliability reflects confidence for pavement structure to remain at the desired serviceability level up to or exceeding the design service life (i.e., the desired initial pavement performance period). The selected design reliability level should consider the uncertainties related to traffic loads, environmental conditions and construction materials to provide a factor of safety into the pavement design. If a pavement structure fails to meet its design service 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 or collector (low traffic) highways/roads. Also repair/resurfacing of rural highways is easier than the repair/resurfacing of urban highways. Similarly, repair/resurfacing of thin surfaced or unsurfaced pavements is easier and less costly than repair/resurfacing of thick/hard surfaces. Therefore, a lower reliability i.e., a higher risk can be considered for secondary/collector highways/roads, rural areas and thin or unsurfaced pavements.

In Manitoba, the selected design reliability has been a function of x-section type (urban versus rural) and highway functional classification, which varied from 80% to 90%. A change was desired to reduce the construction costs for low volume surfaced and unsurfaced roads. Table 6.0.8 presents the desired reliability levels for different highways based on the highway classifications, surface type and highway context i.e., x-section type.

Table 6.0.8: Guidelines for the Selection of Design Reliability, %

Highway Classification	Surface Type	Design Reliability, %	
		Rural x-Section	Urban and Semiurban x-Sections
Freeways	All	95	95
Expressways	All	90	90
All PTHs, and Trade/Commerce Routes, other than freeways and expressways	All	85	90
PR	All	80	85
PR	AST	70	80
PA	AC	70	80
PA	AST	60	70

PTH = Provincial Trunk Highway, PR = Provincial Road, PA = Provincial Access

6.8 Overall Standard Deviation

The overall standard deviation (S_o) reflects the goodness of fits of the AASHTO design equations to AASHO road test data, i.e., the normal variation in pavement performance prediction and the chance variation in the prediction of design traffic loads. The selected design reliability and overall standard deviation account for the combined effect of variation in all design variables (inputs). As such, a best estimate of mean or average value of each design input parameter will provide adequate confidence in the pavement structural design i.e., conservative estimates of design inputs are not required (AASHTO 1993).

Manitoba has been using an overall standard deviation of 0.49 for flexible (AC) and semi-flexible (AST) pavements. For rigid (PCC) and composite pavements, an overall standard deviation of 0.39 was used. However, the estimation or prediction of local traffic loads and determination or estimation of local materials properties have improved significantly over the last several years. Accordingly, an overall standard deviation of 0.45 is recommended for flexible (AC) and semi-flexible (AST) pavements. For the rigid (PCC) and composite pavements, an overall standard deviation of 0.35 is recommended.

6.9 Drainage and Environmental Conditions

In the AASHTO 1993 Design method, the effect of pavement drainage and environmental conditions are accounted for in terms of effective resilient modulus of subgrade soils, subgrade frost heave and swelling consideration, and adjusted structural layer coefficients of granular base and subbase layers for their moisture exposures and drainage qualities. The calculation of resilient modulus of subgrade soils and subgrade frost heave and swelling consideration are discussed in previous chapters (Chapters 3 and 5). AASHTO 1993 design guide provided guideline for adjusting unbound materials structural layer coefficients by using drainage coefficients (m-value). The m-value depends on the drainage quality and percentage of time the layer in question is exposed to moisture approaching saturation moisture level. For example, for a poor drainage quality and exposure to moisture approaching saturation for 5% of time, 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 or very subjective. This may result in a very low effective structural layer coefficient value and lead to a very thick pavement structure, which may be difficult to justify. Another major issue associated with the determination of effective structural layer coefficient value is that the selected m-value from the AASHTO 1993 design guide does not fully account for the seasonal variation of layer stiffness such as the high stiffness of granular subbase and base layers in winter and their low stiffness during the spring melting season in cold climate like Manitoba.

Manitoba did not use the recommended drainage coefficients in the AASHTO 1993 Pavement Design Guide. Instead, Manitoba has been accounting for the impact of pavement drainage and environmental condition by increasing the calculated structural number depending on the highway/road x-section type (urban, semiurban and urban) and surface drainage condition. In this new design manual, the effect of pavement drainage and environmental conditions has been captured using equivalent annual moduli and effective structural layer coefficients of granular materials and the effective value of subgrade resilient modulus. The process of determining the effective value of subgrade resilient modulus has been discussed in the previous chapter. The equivalent annual resilient moduli and effective structural layer coefficients of typical granular materials, considering usual seasonal variation of unbound layer stiffness, mostly dry in summer, partially wet in fall, partially frozen in early winter, fully frozen in winter and saturated in spring, are presented in the next section. The process of determining the equivalent annual elastic (resilient) moduli of other granular materials for different project specific scenarios is also discussed in the next section. Particular attention should be given for areas where groundwater (due to high water table or presence of aquifer) or surface water (due to shallow ditch or lack of adequate drainage) could cause the subbase and base layers remain wet or saturated for more than normal period. The effect of environmental conditions on AC and AST layers should be accounted for using appropriate AC mix design and asphalt binder selection and good construction practices.

6.10 Pavement Layer Materials Properties

The total and individual layer thicknesses of a pavement structure depend on the quality and relative stiffness of materials to be used in road construction. AASHTO 1993 Design Guide provides the required thickness of pavement structure in terms of Structural Number (SN). The SN value is then converted to thickness of different layer materials through use of structural layer coefficients (a-values) of those materials. The structural layer coefficient of a material reflects the relative ability of that material to function as a structural component of a pavement (AASHTO 1993). In the AASHTO 1993 Design Guide, the structural layer coefficients were developed based on empirical relationship between the structural number of a pavement structure and layer thickness.

6.10.1 Structural Layer Coefficient of Asphalt Concrete

Manitoba has been using a structural layer coefficient value of 0.42 for Bituminous B (Bit. B) mixes, which is a typical value used by many other jurisdictions. However, the mineral aggregates in Manitoba's Bit. B mixes have shown to be finely graded as compared to mixes used in many other jurisdictions. Therefore, the structural layer coefficient for Manitoba's typical Bit. B mixes has been lowered to 0.40 based on the laboratory testing at the University of Manitoba (e.g., Harran and Shalaby 2008) for resilient (elastic) modulus on asphalt cores taken from highway/road projects. For SuperPave (SP) AC mixes, which are typically stiffer than Bit. B mixes, higher structural layer coefficient values than Bit. B mixes are recommended.

The natural aggregates in the South-Western Manitoba and some other locations across the province can be softer, lighter and highly moisture absorptive than typical good to excellent quality aggregate materials. They usually contain shale and other soft/lightweight particles. The AC mixes with such aggregate particles experience stripping issues in addition to other distresses. Despite using anti-stripping agents in AC mixes, worse pavement performance has been an issue in those areas as compared to other areas of the province. Therefore, a lower structural layer coefficient value should be used for an AC mix that contains aggregates with low specific gravity and high percentage of water absorption.

Technically, an effective elastic (resilient) modulus and structural layer coefficient of AC layer should be used in the design considering variation of stiffness from month to month or season to season. The temperature sensitivity of the asphalt mixes makes it difficult to establish representative value for any month or even for a given day. The accuracy of such effective modulus would be poor, if possible to establish at all. Therefore, the structural layer coefficient value at standard temperature (20°C), as recommended in the AASHTO 1993 Design Guide, has been considered as the most suitable and conservative option.

The typical values of structural layer coefficients for different AC mixes are presented in Table 6.0.9. The designer should refer to department's relevant engineering standard for possible changes to these tabulated values. The structural value of thin (10 to 20 mm in thickness) chip seals (AST) for semi-flexible pavement is small and can be ignored for pavement layer thickness determination.

Table 6.0.9: Structural Layer Coefficients of AC Mixes

AC Mix Type	Layer Coefficient (a_1) Value
Bit. B (**Note 1**)	0.40
Bit. B (**Note 2**)	0.36
SP19.0 (**Note 1**)	0.44
SP 19.0 (**Note 2**)	0.40
SP12.5 (**Note 1**)	0.42
SP 12.5 (**Note 2**)	0.38

Note 1: Mixes with good quality coarse aggregate and fine aggregate each having a bulk specific gravity (oven dry basis) of ≥2.60 and a water absorption of ≤2.50%.

Note 2: Mixes with fair quality coarse aggregate and/or fine aggregate each having a bulk specific gravity (oven dry basis) of ≥2.50 to <2.60 and/or water absorption of >2.50% to ≤3.50%.

6.10.2 Structural Layer Coefficients of Granular Base and Subbase

After extensive laboratory and field testing, trials and evaluation, Manitoba developed new specifications for granular base and subbase materials, which are stiffer and more stable with better drainage quality than the historically used A-base, C-base (subbase) and Modified C-base (granular fill) materials. Several tests for resilient modulus (M_R) and permeability were conducted by the Pavement Research Group at the University of Manitoba (Soliman and Shalaby, 2011; Soliman and Shalaby 2015; Ahmeduzzaman and Shalaby, 2016; and Mneina et al. 2018). Extensive CBR tests were conducted by the department's Central Laboratory on base and subbase samples collected from the project sites during the research, trial and evaluation, and the implementation phases of these new specifications. Tests were also conducted on previously used granular base and subbase materials.

For the unbound granular base and subbase materials, AASHTO 1993 Design Guide has provided equations and charts to estimate the layer coefficients from M_R, CBR, R and Texas Triaxial values. Based on the results of M_R, CBR and permeability tests, field evaluation of stability under compaction and traffic, layer stiffness with FWD deflection testing and drainage qualities, and the correlation charts provided in the AASHTO 1993 Design Guide, representative M_R values of typical granular base, subbase and fill materials for summer in-situ condition have been established.

The seasonal factors for the variation of base/subbase/fill layers resilient moduli in a year depend on: a) highway context (i.e., urban, semiurban and rural x-sections) that affects effective drainage; b) drainage quality of the material; and c) depth of layer (e.g., base versus subbase). It should be noted that base layer is close to the surface, and as a result, it is more exposed to moisture and freeze/thaw weakening than the subbase layer. The equivalent annual resilient moduli and the effective structural layer coefficients of currently used granular materials have been developed considering typical month to month variation of moisture and stiffness of different materials and above listed variables.

As mentioned in an earlier chapter, Manitoba has developed seasonal factors for subgrade soil M_R 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 previously used A-base layer modulus in different time of the year. The seasonal factors for resilient moduli variation of A-base materials and moisture susceptibility of new granular materials to stiffness variation as compared to the A-base material were also used to establish the seasonal factors for moduli variation of new granular materials. The current summer representative resilient moduli, equivalent annual resilient moduli, and the effective structural layer coefficient values of typical granular materials are presented in Table 6.10.

Table 6.0.10: Equivalent Annual Moduli and Layer Coefficients of Typical Granular Base and Subbase Materials

	Equivalent Annual M_R, MPa (psi)			**Effective Structural Layer Coefficients**		
Granular Material (Summer Rep. M_R)	**Rural**	**Semi-urban**	**Urban**	**Rural**	**Semi-urban**	**Urban**
GBC- S (145 MPa) (**Note 1**)	152 (21,990)	152 (21,990)	138 (20,070)	0.104	0.104	0.094
GBC- S (130 MPa) (**Note 2**)	136 (19,720)	136 (19,720)	124 (17,990)	0.092	0.092	0.083
GBC- I (200 MPa) (**Note 1**)	224 (32,490)	224 (32,490)	206 (29,810)	0.146	0.146	0.137
GBC- I (170 MPa) (**Note 2**)	190 (27,610)	190 (27,610)	175 (25,340)	0.129	0.129	0.120
GBC- II (180MPa) (**Note 1**)	202 (29,240)	202 (29,240)	185 (26,830)	0.135	0.135	0.126
GBC- II (155 MPa) (**Note 2**)	174 (25,180)	174 (25,180)	159 (23,100)	0.119	0.119	0.110
GBC- M (170 MPa) (**Note 1**)	190 (27,610)	190 (27,610)	175 (25,340)	0.129	0.129	0.120
GBC- M (150 MPa) (**Note 2**)	168 (24,360)	168 (24,360)	154 (22,360)	0.115	0.115	0.106
GSB- C (110 MPa) (**Note 1**)	119 (17,310)	107 (15,500)	107 (15,500)	0.123	0.112	0.112
GSB- C (100 MPa) (**Note 2**)	109 (15,730)	97 (14,090)	97 (14,090)	0.114	0.103	0.103
GSB- F (90 MPa) (**Note 1**)	98 (14,160)	88 (12,680)	88 (12,680)	0.103	0.092	0.092
GSB- F (80 MPa) (**Note 2**)	87 (12,590)	78 (11,280)	78 (11,280)	0.092	0.081	0.081
CR- M50 (250 MPa) (**Note 1**)	231 (33,510)	219 (31,780)	219 (31,780)	0.188	0.183	0.183
CR- M50 (210 MPa) (**Note 2**)	194 (28,145)	184 (26,695)	184 (26,695)	0.171	0.166	0.166
CR- M100 (**Note 1**)				0.169		
CR- M100 (**Note 2**)				0.154		
CR- M125 (**Note 1**)				0.160		
CR- M125 (**Note 2**)				0.146		

Note 1: Good quality coarse aggregate and fine aggregate each having a bulk specific gravity (oven dry basis) of ≥2.60 and a water absorption of ≤2.50%.

Note 2: Fair quality coarse aggregate and/or fine aggregate each having a bulk specific gravity (oven dry basis) of ≥2.50 to <2.60 and/or water absorption of >2.50% to ≤3.50%.

The seasonal factors for resilient moduli variation of new granular base and subbase materials will be confirmed with additional field testing of these materials from sources around the province and updated structural layer coefficient values will be provided in a relevant engineering standard of the department. The designers should refer to the department's latest engineering standard for the available updated values.

It should be noted here that the effect of climatic variation among different climate zones in Manitoba are not considered in the equivalent annual resilient modulus calculation because of low moisture holding capacity of new granular subbase and base materials for freezing into solid state and their ability to drain entrapped or entrained moisture rapidly during freeze-thaw cycles and spring thawing seasons.

6.10.3 Calculation of Effective Layer Coefficients of Granular Materials

For non-typical (special) scenarios such as for locations where granular fill, subbase and base layers are exposed to moisture for a longer period than typical spring and/or fall due to high water table, standing water in ditch or along roadside, flooding, presence of aquifers, etc., site-specific equivalent annual resilient moduli for granular materials for that location should be calculated to determine the effective structural layer coefficients.

The Quintus and Killingsworth (1997) provided equations (Equations 6.5 and 6.6) to calculate the equivalent annual resilient moduli of unbound base and subbase layers. It is recommended that the calculated equivalent annual resilient modulus value using these equations should be used to determine the minimum thickness of AC layer following the AASHTO 1993 Design Guide approach to limit the tensile strain to an acceptable limit. Since the base and subbase layers act as intermediate foundations for a surface layer, the equivalent annual M_R value calculated using these equations account for the relative damage due to seasonal variation of M_R values of these granular material layers.

$$U_f = 1.885 * 10^3 * (M_R)^{-0.721} \tag{6.5}$$

$$M_{Rea} = \frac{[\Sigma(U_{fi})*(M_{Ri})]}{\Sigma(U_{fi})} \tag{6.6}$$

where,

U_f = damage factor for a given M_R,

U_{fi} = damage factor for season i,

M_{Ri} = resilient modulus in season i (psi), and

M_{Rea} = equivalent annual resilient modulus of granular material

The calculated (using above equations) equivalent annual resilient moduli of unbound granular base and subbase materials can be used to estimate the effective structural layer coefficients of these materials if a large modulus ratio between successive layers does not occur. A high modulus ratio between successive layers can result in a high tensile stress at the bottom of the base or subbase layer, which can loosen the base or subbase material due to their decompaction (Quintus and Killingsworth, 1997). A loosened layer material will then exhibit a lower resilient modulus. Based on the design M_R values of Manitoba's granular base and subbase materials, the modulus ratio between base and subbase is not high (i.e., not >3). However, the modulus ratio between granular subbase and native subgrade could be high (>3). The modulus ratio between subbase and subgrade are currently ignored by Manitoba for a cost-effective use of high quality subbase material. The geotextile fabrics placed between subbase and subgrade may reduce potential decompaction of subbase layer. However, consideration should be given to stabilize the top 300 mm of weak subgrade, having a resilient modulus value of less than 35 MPa, using cementitious material to reduce the modulus ratio between subbase and the underlying subgrade, where feasible.

The effective structural layer coefficient of a granular base can be calculated using Equation 6.7 (Quintus and Killingsworth, 1997). The effective structural layer coefficient of any granular subbase can be calculated using Equation 6.8 from AASHTO 1993 Design Guide.

$$a_2 = 0.249 * (log_{10} M_{Rea}) - 0.977 \tag{6.7}$$

$$a_3 = 0.227 * (log_{10} M_{Rea}) - 0.839 \tag{6.8}$$

where,

a_2 = base layer coefficient,

a_3 = subbase layer coefficient, and

M_{Rea} = equivalent annual resilient modulus of base or subbase (psi)

Example of Equivalent Annual M_R and Effective Layer Coefficient Calculation

Table 6.0.11 shows an example of equivalent annual M_R and effective structural layer coefficient calculation for a granular base material.

Table 6.0.11: Example of Seasonal Factors and Equivalent Annual Modulus

Summer M_R, psi =	**29,008 (200 MPa)**		
Month	**Seasonal Factors**	**Seasonal M_R, psi**	**Seasonal Rel. Damage**
January	3.00	87,022.80	0.51741
February	3.00	87,022.80	0.51741
March	0.70	20,305.32	1.47749
April	0.70	20,305.32	1.47749
May	0.80	23,206.08	1.34188
June	0.90	26,106.84	1.23263
July	1.00	29,007.60	1.14246
August	1.00	29,007.60	1.14246
September	1.00	29,007.60	1.14246
October	0.80	23,206.08	1.34188
November	1.00	29,007.60	1.14246
December	3.00	87,022.80	0.51741
Sum Product (U_f *M_R)			422100.82
Sum U_f			12.99
Equivalent M_R, psi			32,486
Equivalent M_R, MPa			224.0
Effective Structural Layer Coefficient			0.146

Conversion: 1 MPa = 145.038 psi

The summer M_R represents the resilient modulus determined in the laboratory at typical in-situ density (say, 98% of the maximum dry density) and corresponding (i.e., optimum) moisture content or the estimated M_R from the soaked CBR value corresponding to 2.54 mm (0.1 in.) penetration with specimen(s) prepared at the same density and moisture content as the resilient modulus test. The resilient modulus can be estimated from CBR using the following equation (Christopher et al. 2006).

$$M_R = 17.6 * (CBR)^{0.64} \quad (6.9)$$

where,

M_R = resilient modulus of granular material, MPa

CBR = California bearing ratio in soaked condition, % (as per AASHTO T193)

Table 6.0.12 presents a general guideline for selecting the seasonal factors for resilient modulus variation, based on moisture content and freezing as well as thawing conditions of a well graded granular material, for which no equivalent annual modulus and effective structural layer coefficient are provided in this manual. These factors are not applicable to crushed rock material with a maximum aggregate size of 50 mm or larger. These factors should be adjusted based on site specific exposure to moisture and freezing/thawing conditions, where applicable.

Table 6.0.12: Seasonal Factors for M_R Variation of Granular Materials

Materials Type/Condition	**Seasonal Factors**		
	PI<4% and Fines Content ≤ 9%	**PI<4% and Fines Content >9% to ≤ 13%**	**PI ≤7% and Fines Content >13 to ≤17%**
Fairly dry (moisture content ≤OMC)	1.00	1.00	1.00
Moist	0.80	0.70	0.60
Frozen	3.00	3.00	3.00
Thawed (partially saturated)	0.70	0.60	0.50

Fines = Material passing the 0.075 mm sieve

6.11 Pavement Structure for New Construction and Full Depth Reconstruction

AASHTO 1993 Pavement Design Guide provides the required thickness of a pavement structure in terms of design (also called total or overall) structural number (SN), which is required to withstand traffic load repetitions over the design service life for given subgrade stiffness, pavement serviceability and reliability. It is calculated using the following formula (AASHTO 1993):

$$log_{10}(W_{18}) = Z_R * S_o + 9.36 * log_{10}(SN+1) - 0.20 + \frac{log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32 * log_{10}(M_R) - 8.07 \quad (6.10)$$

where,

W_{18} = number of standard 80 kN (18,000 lb) load repetitions (ESALs) over the design service life

Z_R = standard normal deviate (depends on design reliability)

S_o = overall standard deviation (0.45)

SN = overall (Design) structural number, inches

ΔPSI = serviceability loss due to traffic loads

M_R = subgrade resilient modulus, psi

The standard normal deviate value should be selected from Table 6.0.13 based on the selected design reliability.

Table 6.0.13: Standard Normal Deviate Values

Design Reliability, %	Standard Normal Deviate, Z_R
95	-1.645
90	-1.282
85	-1.037
80	-0.841
75	-0.674
70	-0.524
60	-0.253
50	0.0

Manitoba has been using the AASHTO DARWin software to determine the design structural number. In the absence of DARWin software (which is no longer supported by AASHTO), Equation 6.10 can be solved for the structural number using simple computer program and macro (e.g., MS Excel, MS Access and MATLAB). Alternatively, the design chart (Part II, Chapter 3, Figure 3.1) in the AASHTO 1993 Design Guide can be used to determine the structural number. The total (design) SN then should be converted to layer thickness of different materials to be used in the actual construction.

6.11.1 Selection of Layer Thicknesses

Using Equation 6.11, the design SN can be converted into thicknesses of different layer materials using the effective structural layer coefficient (a) values of the materials that are to be used in the actual construction of pavements. It should be noted that drainage coefficient (m) in not required when using the effective layer coefficients.

$$SN_{dgn} = \sum D_i a_i \tag{6.11}$$

where,

D_i = thickness of layer i (1, 2, 3, 4………..)

a_i = structural layer coefficient value of layer i (1, 2, 3, 4………..)

For calculating the SN of AC layers, the effective thickness of the bottom layer of AC should be taken as the selected thickness minus 12.5 mm (considering 12.5 mm loss for levelling the unbound material layer surface).

6.11.2 Minimum Thickness of Asphalt Concrete Layer(s)

In general, the minimum thickness of AC layer(s) should be determined based on the layered design analysis (as described in the next section) using the selected design reliability of the pavement structure for each highway/road section. However, for a highway/road section with 20-years design of ESALs <3.0 million, the minimum thickness of the AC layer can be determined following layered design analysis at 50% design reliability. For a highway/road section with 20-years design ESALs of 3.0 million or greater, the minimum thickness of AC layer can be selected following layered design analysis at 50% design reliability if the available budget is inadequate to construct the full AC thickness at the required design reliability of the full pavement structure. If the subgrade soil on a highway/road section is classified as highly or very highly frost susceptible, the minimum thickness of the AC layer can also be determined following layered design analysis at 50% design reliability, regardless of design traffic loads, to increase the total thickness of pavement structure with additional granular layer and maximize frost protection. The serviceability loss due to frost heave should be ignored for the determination of minimum AC layer thickness.

In no case, should the thickness of AC layer (including 12.5 mm for levelling) be less than the minimum thickness specified in Table 6.0.14.

Table 6.0.14: The Absolute Minimum AC Layer Thickness

Highway Loading Class	20-Years Design ESALs	Minimum Surface Layer Thickness
All, except RTAC	<150,000	AST (double chip seals) or 80 mm AC (50 mm for spring weight restricted highways/roads)
All, except RTAC	150,000 to <300,000	AST (double chip seals) or 85 mm AC (60 mm for spring weight restricted highways/roads- ***Note 1***)
All, except RTAC	300,000 to <1,000,000	90 mm AC
All, except RTAC	1,000,000 to <3,000,000	100 mm AC
All, except RTAC	≥3,000,000	Based on layered design analysis
RTAC	N/A	150 mm AC

Note 1: Generally, a spring weight restricted AC pavement is not recommended because of added costs as compared to double chip seals and potential break-ups due to fully loaded trucks.

The thickness of AC layer on any RTAC route should be a minimum of 150 mm, regardless of the outcome from the layered design analysis or Table 6.0.14. This minimum AC thickness is required to withstand increased damage caused by axles having wide-base single tires as compared to the damage caused by axles having standard dual tires. It should be noted here that axles having wide-base single tires are allowed the same weights as the standard axles having dual tires for travel on RTAC routes only. On class A1 and class B1 highways, axles having wide-base single tires are allowed reduced weights than that allowed for the standard axles having dual tires travelling on these highways.

6.11.3 Minimum Thickness of Granular Layer(s)

The thickness of each of the granular base and subbase layers should not be less than or more than three times the thickness of immediately overlying layer material. The total thickness of the granular base and subbase layers including any granular fill should also be a minimum of 300 mm for any subgrade soil having a resilient modulus value of ≤50 MPa even though the design calculation indicates a lesser requirement. For subgrade/fill having a resilient modulus value of >50 MPa to ≤70 MPa, a 200 mm thick base/subbase should be placed, as a minimum. The subbase requirements can be omitted if the nominal maximum aggregate size of the underlying granular fill material is less than twice the nominal maximum aggregate size of the granular base layer.

For grade widening of an existing roadway, the total thickness of the pavement structure on the new grade should be the same as or thicker than the pavement structure on the adjacent existing lane for the continuity of lateral drainage of moisture from the entire pavement structure x-section. The layer thickness should be selected (i.e., thicker subbase layers and thinner base and/or AC layers) to limit the increase in cost to meet this requirement.

6.12 Layered Design Analysis and Design

As mentioned in a previous chapter, pavements are generally layered structures, which consist of several layers of different materials. Each underlying layer acts as the foundation for the overlying layer(s). The base layer acts as a foundation for the surface layer, the subbase layer acts as the foundation for surface and base layers, the subgrade acts as the foundation for all the layers overlying it. As such, the SN of the surface layer should be determined using equivalent annual resilient modulus of base layer while the total SN of surface and base layers should be determined using the equivalent annual resilient modulus of subbase layer. The total SN of all layers should be determined using the effective resilient modulus of subgrade layer. The equivalent annual modulus of base layer should be limited to 250 MPa for determining the SN of the surface layer. When CR- M50 is used as a subbase material, GBC- I should be used as a base layer material, wherever possible.

The layer thickness determined through the process outlined above reflects the required minimum thickness of each layer, with the exception as outlined in Sections 6.11.2 and 6.11.3. However, as mentioned earlier, the thickness of any layer should not be less than or more than three times the thickness of the layer immediately overlying that layer. If the effective structural layer coefficients of base and subbase layers are known (e.g., values in Table 6.0.10), the equivalent annual resilient modulus of base and subbase layers can be backcalculated using from Equations 6.7 and 6.8, respectively.

6.13 Design Examples

<u>Example 1 (Low Traffic Loads): Highway Information</u>

a) Highway: A provincial undivided two-lane collector highway in Northern Region (climate zone 3)
b) Highway loading classification: B1
c) Traffic volume: AADT of 1,000 with 100 trucks per day (2-way) and 2% annual growth rate
d) Design service life: 20 years

e) Subgrade type and resilient modulus: Sandy clay with a summer M_R value of 40 MPa

f) Subgrade soils frost heave potential: Negligible

g) Drainage and environmental conditions (highway context): Rural

h) Pavement layer materials: Bit. B surface, GBC- M base and GSB- C subbase

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.5 and TEF of 1.22 = 541,000
Subgrade effective resilient modulus = 35.5 MPa
Design reliability = 80%
Initial serviceability index = 4.2 (assuming two lifts of AC)
Terminal serviceability index = 2.2
Serviceability loss due to traffic (no loss due to frost) = 4.2 - 2.2 = 2.0
Overall standard deviation = 0.45
Structural layer coefficients: Bit. B = 0.40, GBC- M = 0.129 and GSB- C = 0.123

Design SN and Layer Thickness

The calculated total (design) SN_{dgn} = 83.1 mm
SN of surface layer using equivalent annual modulus of GBC- M (190 MPa) and 50% design reliability = 37.9 mm
Determine Bit. B layer thickness = 37.9/0.40 = 95 mm; say, **100 mm thick Bit. B** layer will be placed
Effective SN of surface layer (SN_1) = (100- 12.5) * 0.40 = 35.0 mm
Select base (GBC- M) layer thickness; say, **150 mm thick GBC- M** layer will be used
SN of base layer (SN_2) = 150 * 0.129 = 19.3 mm
SN of subbase layer (SN_3) = SN_{dgn} – (SN_1 + SN_2) = 83.1 – (35.0 + 19.3) = 28.8 mm
The required thickness of subbase (GSB- C) = 28.8/0.123 = 234 mm; say, **250 mm thick GSB- C** layer will be used.

Example 2 (High Traffic Loads/Layered Design Analysis): Highway Information

a) Highway: A provincial four-lane divided expressway (NHS- Core route) in Capital Region (climate zone 1A)

b) Highway loading classification: RTAC (NHS- Core route)

c) Traffic volume: AADT of 7,000 with 1,400 trucks per day (1-way) and 1.4% annual growth rate

d) Design service life: 20 years

e) Subgrade type and resilient modulus: High plastic clay with a summer M_R of 18.0 MPa (2,610 psi)

f) Subgrade soils frost heave potential: Negligible

g) Drainage and environmental conditions (highway context): Rural

h) Pavement layer materials: SuperPave AC, GBC- I base and CR-M50 subbase

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.9 and TEF of 1.739 = 18,300,000
Subgrade effective resilient modulus = 15.5 MPa (2,248 psi)
Design reliability = 90%
Initial serviceability index = 4.4 (assuming four lifts of AC) (adjust the initial serviceability index based on the actual number of lifts, if required)
Terminal serviceability index = 2.5
Serviceability loss due to traffic (no loss due to frost) = 4.4 - 2.5 = 1.9
Overall standard deviation = 0.45
Structural layer coefficients: SP12.5 AC = 0.42, SP19 AC = 0.44, GBC- I = 0.146 and CR-M50 = 0.188
Equivalent annual modulus of GBC- I = 224 MPa (32,490 psi)
Equivalent annual modulus of CR-M50 = 231 MPa (33,510 psi)

Design SN and Layer Thickness

SN of AC layers using equivalent annual modulus of base layer (224 MPa) = 80.0 mm
Assume that 40 mm thick SP12.5 AC will be used as top surface and the rest of the AC will be SP19.0
SN of SP12.5 AC layer = 40 * 0.42 = 16.8 mm
Thickness of SP19.0 AC layer = (80.0 – 16.8)/0.44 = 144 mm; say, 160 mm thick SP19.0 AC layer (including levelling) will be used
A **200 mm thick AC (40 mm thick SP12.5 and 160 mm thick SP19.0)** will be required
Effective SN of SuperPave AC layers (SN_1) = 40*0.42 + (160 - 12.5) * 0.44 = 81.7 mm
SN of AC and base layers ($SN_{1,2}$) using equivalent annual modulus of subbase layer (231 MPa) = 79.1 mm

SN of base layer = 79.1 – 81.7 = <0

Thickness of GBC- I layer: **200 mm thick GBC- I** layer will be used to meet the AC to base layers thickness ratio

Effective SN of GBC-I layer (SN_2) = 200*0.146 = 29.2 mm

Total SN of all layers (SN_{dgn}) using the effective modulus of subgrade (15.5 MPa) = 190.5 mm

SN of subbase layer (SN_3) = SN_{dgn} - (SN_1 + SN_2) = 190.5 - (81.7 + 29.2) = 79.6 mm

The required thickness of subbase (CR- M50) layer = 79.6/0.188 = 423 mm; say, **450 mm thick CR- M50** (three lifts with 150 mm per lift) layer will be used.

Example 3 (Frost Susceptible Subgrade): Highway Information

a) Highway: A provincial two-lane divided arterial highway in Western Region (climate zone 1A)
b) Highway loading classification: RTAC
c) Traffic volume: AADT of 5,800 with 400 trucks per day (2-way) and 1.1% annual growth rate
d) Design service life: 20 years
e) Subgrade type and resilient modulus: Sandy Silt (ML) with a summer M_R of 50 MPa and 38% particles smaller than 0.02 mm.
f) Subgrade soils frost heave potential: Severe frost heave with an average interval of 75-100 m and frost heave occurs every year
g) Drainage and environmental conditions (highway context): Rural
h) Pavement layer materials: Bit. B surface, GBC- I base and GSB- C subbase. Aggregates exhibit low specific gravity (<2.60) and high water absorption (>2.50%).

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.50 and TEF of 1.395 = 2,265,000

Subgrade effective resilient modulus = 43 MPa (6,235 psi)

Design reliability = 85%

Initial serviceability index = 4.3 (assuming three lifts of AC)

Terminal serviceability index = 2.5

Average frost heave rate = 8 mm/day

Average frost depth = 2.0 m

Assuming a total thickness of pavement structure as 750 mm, frost penetration into subgrade = 2.0 - 0.75 = 1.25 m (adjust as needed based on the calculated total thickness)
Maximum serviceability loss due to frost = 1.3128 * 1.25 = 1.64 (using Equation 6.2)
Frost probability = 50 % (using Equation 6.3)
Serviceability loss due to frost heave = 0.69 (using Equation 6.1)
Serviceability loss due to traffic loads = 4.3 - 2.5 – 0.79 = 1.01
Overall standard deviation = 0.45
Structural layer coefficients: Bit. B (with fair quality aggregates) = 0.36, GBC- I (with fair quality aggregates) = 0.129 and GSB- C (with fair quality aggregates) = 0.114
Equivalent annual modulus of GBC- I (with fair quality aggregates) = 190 MPa (27,610 psi)

Design SN and Layer Thickness

The calculated total (design) SN_{dgn} = 120.1 mm
SN of surface layer using equivalent annual modulus of GBC- I (190 MPa), 50% reliability and ignoring serviceability loss due to frost heave = 48.9 mm
Bit. B layer thickness = 48.9/0.36 = 136 mm; say, **150 mm thick Bit. B** layer will be used (including levelling)
Effective SN of surface layer (SN_1) = (150- 12.5) * 0.36 = 49.5 mm
Select base (GBC- I) layer thickness; say, **200 mm thick GBC- I** layer will be used
SN of base layer (SN_2) = 200 * 0.129 = 25.8 mm
SN of subbase layer (SN_3) = SN_{dgn} – (SN_1 + SN_2) = 120.1 – (49.5 + 25.8) = 44.8 mm
The required thickness of subbase (GSB- C) = 44.8/0.114 = **393 mm;** say, **400 mm thick GSB- C** layer will be used.
Check: Total thickness = 150 + 200 + 400 = 750 mm

The design total thickness of pavement structure matches with the assumed total thickness. Therefore, no reanalysis is required in this case. However, consideration should be given to increase the total thickness to 1.0 m i.e., 50% of the frost depth (70% in the case of NHS Core and Intermodal routes) in this case of severe frost heave issue (the same applies to very severe frost heave issues). Replacement of a part of the native subgrade in the sub-cut with select granular fill could be an economic option (as the select granular fill can be considered a subbase layer) because the grade width and height will remain unchanged or even could be reduced. Alternatively, the top 300 mm of subgrade could be stabilized with cementitious material.

6.14 Pavement Analysis and Design for Frost Heave Management

Although removal of frost susceptible soils and replacement with non-frost susceptible materials or constructing a thick embankment using non-frost susceptible materials are the desirable options, they may not be economically feasible at most of the highway locations. The following practices to manage the frost heave issues, instead of costly treatment or control of frost heave, are recommended for highway locations where increased granular (fill/subbase) thickness to mitigate frost issues are not feasible:

i) Remove frost susceptible soils and replace with non-frost susceptible granular fill or subbase from isolated areas with severe and very severe frost heave issues. The total thickness of pavement structure at these isolated areas, including any additional non-frost susceptible granular subbase and/or fill materials, should be at least 50% of the frost penetration depth at the project location, except for the NHS Core and Intermodal routes. For the NHS Core and Intermodal routes, the total thickness of pavement structure at these isolated areas, including any additional non-frost susceptible granular subbase and/or fill materials, should equate to at least 70% of the frost penetration depth at the project location. As indicated in an earlier chapter, the use non-frost susceptible granular fill, instead of the excavated in-situ frost susceptible material, in subgrade sub-cut (typically 0.6 m) will be a prudent approach to avoid wider grade with thicker pavement structures.

ii) Construct pavement structure on the entire area with design for a reduced service life, which is developed through the following procedure:

   1) Determine the required pavement structure in terms of total SN for 20 years performance period without considering the serviceability loss due to frost heave i.e., using the 20 years design traffic loads, subgrade M_R, reliability and total serviceability loss.

   2) Assume the expected service life and the total thickness of the pavement structure under both traffic loads and frost heave conditions. A lower performance period is expected for greater loss (based on frost severity and probability) due to the frost heave issues.

   3) Calculate the individual serviceability loss due to frost heave and traffic loads.

4) Calculate the required SN for the expected (i.e., reduced) service life of the initial pavement for the serviceability loss due to traffic alone.

5) Repeat Steps #2 to #4 until the calculated SN due to traffic alone matches with the 20-year design SN determined in Step #1.

6) The service life (i.e., performance period) of the initial pavement structure should not be less than 10 years. If the calculated service life of the initial pavement structure in Step #5 is less than 10 years, determine the required SN for a 10 years service life of the initial pavement structure under both traffic loads and frost heave conditions. This can be done through the following procedure:

   a) Set the design service life of the initial pavement structure as 10 years (or more, if desired)

   b) Assume a higher total thickness of pavement structure than that was assumed in Step # 2 of the last iteration.

   c) Follow Steps # 3 and 4 to determine the design SN.

7) Confirm that the total thickness of all layers matches with the assumed total thickness in Step # 2 (last iteration) or # 6(b), as applicable.

8) Indicate the timing (year) of the expected overlay requirement in the Pavement Structure and Surfacing Design Memo (PSSDM) or pavement design report.

iii) Overlay or mill and overlay the pavement when the serviceability falls below the desirable level.

<u>Example 4 (Frost Susceptible Subgrade- Reduced Service Life): Highway Information</u>

Refer to Design Example #3

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.50 and TEF of 1.395 = 2,265,000
Subgrade effective resilient modulus = 43 MPa
Design reliability = 85%
Initial serviceability index = 4.3 (assuming three lifts of AC)
Terminal serviceability index = 2.5
Average frost heave rate = 8 mm/day

Average frost depth = 2.0 m
Frost probability = 50 % (using Equation 6.3)
Overall standard deviation = 0.45
Structural layer coefficients: Bit. B (with fair quality aggregates) = 0.36, GBC- I (with fair quality aggregates) = 0.129 and GSB- C (with fair quality aggregates) = 0.114
Equivalent annual modulus of GBC- I (with fair quality aggregates) = 190 MPa

Design SN and Layer Thickness

The calculated total SN for 20 years' service life without considering frost heave = 102.1 mm

Trial 1
Assume the total thickness of pavement structure for the design SN of 102.1 mm; say 600 mm.
Assume the expected service life of this pavement structure under both traffic loads and frost heave conditions; say, 12 years (1.3 million ESALs).
Depth of frost penetration into subgrade = 2.0 - 0.60 = 1.40 m (adjust if the total thickness of pavement is changed)
Maximum serviceability loss due to frost = 1.3128 * 1.40 = 1.84 (using Equation 6.2)
Serviceability loss due to frost heave = 0.78 (using Equation 6.1)
Serviceability loss due to traffic loads = 4.5 - 2.5 - 0.78 = 1.02
Calculated SN for a serviceability loss of 1.02 (due to traffic) and the calculated traffic loads of 1.3 million ESALs = 108.6 mm, which is >102.1 mm (i.e., design SN). Therefore, pavement structure with a design SN of 102.1 mm is not adequate for the assumed performance period of 12 years under both traffic loads and frost heave conditions.

Trial 2
Assume the total thickness of pavement structure for the design SN of 102.1 mm; say, 600 mm.
Assume the expected service life of this pavement structure under both traffic loads and frost heave conditions; say, 10 years (1.07 million ESALs).
Depth of frost penetration into subgrade = 2.0 - 0.60 = 1.40 m (adjust if the total thickness of pavement is changed)
Maximum serviceability loss due to frost = 1.3128 * 1.40 = 1.84 (using Equation 6.2)
Serviceability loss due to frost heave = 0.73 (using Equation 6.1)
Serviceability loss due to traffic loads = 4.5 - 2.5 – 0.73 = 1.07

Calculated SN for a serviceability loss of 1.07 (due to traffic) and traffic loads of 1.07 million ESALs = 103.1 mm, which is slightly over the design SN of 102.1 mm. Since the acceptable minimum design service life is 10 years, a design SN_{dgn} of 103.1 mm should be selected.
The service life of this pavement structure under both traffic loads and frost heave conditions is expected to be 10 years.

Thickness Calculation and Check
From the Design Example #3, effective SN of 150 mm AC surface layer (SN_1) = (150- 12.5) * 0.36 = 49.5 mm and SN of 200 mm GBC- I layer (SN_2) = 200 * 0.129 = 25.8 mm
SN of subbase layer (SN_3) = SN_{dgn} – (SN_1 + SN_2) = 103.1 – (49.5 + 25.8) = 27.8 mm
The required thickness of subbase (GSB- C) = 27.8/0.114 = 244 mm; say, **250 mm thick GSB-C** layer will be used.
Check: Total thickness of pavement structure = 150 + 200 + 250 = 600 mm. It matches with the assumed total thickness; so, the design is acceptable.

6.15 Design Adjustment for Organics in Subgrade or Embankment Soils

All in-situ and borrowed subgrade/embankment soils should be tested for stiffness (resilient modulus or CBR) and organic contents before providing the final design of pavement structures. The seasonal factors for the resilient modulus variation should be adjusted considering the organic contents and moisture susceptibility of the materials under consideration. For example, a seasonal factor of 0.40 (instead of typical 0.5) should be used for spring months and wet or saturated subgrade conditions if the organic contents in soils exceed 6% but does not exceed 10%. A seasonal factor of 0.25 should be used for spring months and wet or saturated subgrade conditions if the organic contents in soils exceed 10%. If no stiffness data is available, the preliminary design can be developed using typical modulus value of the predominant soil types and increasing the calculated total SN with an adjustment factor (apply the largest increase) depending on the percentage, depth and extent of organics in in-situ subgrade soils as shown in Table 6.0.15. If organic contents in the borrowed soils exceed 3% (and do not exceed 6%), the preliminary design total SN should be increased by 20%.

Table 6.0.15: Adjustment for Organics in In-Situ Subgrade Soils (MIT 2009)

Subsoil Zone	Depth Below Design Subgrade (mm)	Organic Content (%)	Note	Extent of Organics	SN Adjustment or Action
Sub-cut	0 – 600	4-6	A	Discontinuous, randomised layers	10%
		4-6	B	Continuous layers ≥100 mm thick	20%
		7-10	A	Discontinuous, randomised layers	20%
		7-10	B	Continuous layers ≥100 mm thick	40%
		11 or more		At least some distinct deposits	Excavate
Below Sub-cut	600 – 1200	7-10	A	Discontinuous, randomised layers	10%
		7-10	B	Continuous layers ≥200 mm thick	20%
		11 or more	A	Discontinuous, randomised layers	30%
		11 or more	B	Continuous layers ≥200 mm thick	40%
		11 or more		Deposits ≥300 mm thick	Excavate
	1200 – 1800	11 or more		Continuous layers ≥200 mm thick	20%
		11 or more		Deposits ≥300 mm thick	40%

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

6.16 Minimum Pavement Structure for a Non-Spring Weight Restricted Highway

An analysis by Manitoba has shown that the traffic loads in spring can cause five times increased damage to pavement as compared to the damage caused by the same amount of loads in summer condition. The volume of truck traffic on some collector and access roads can be too low to provide sufficient pavement structures that can support fully loaded trucks without causing intolerable damage to the pavements during the spring thawing period. As such, Manitoba has been constructing a certain minimum pavement structure on many highway sections, despite the calculated service life ESALs provide thinner pavement structures. This practice has been followed to avoid imposing any restrictions on the allowable axle weights during the spring thawing period on selected highways/roads.

In the past, the department used a minimum design ESALs of 370,000 calculated based on the Modified Shell Method, which corresponds to a design BBR value of <1.50 mm, for the design of a non-spring weight restricted highway. This design ESALs equated to approximately 25 trucks per day on the design lane of those collector and access roads. The department has also been providing pavement design for turning (acceleration, deceleration, cut-off) lanes for a minimum of 25 trucks/day design lane traffic even when the actual truck volumes were less than 25 trucks/day. These pavement structures have shown to perform satisfactorily.

The only limitation of the above stated approach is that it does not account for the variation of traffic loads i.e., the same design was provided whether there were 10 trucks or 25 trucks on a road (for a given subgrade). In fact, it is difficult to estimate the required minimum pavement structure that can withstand few trucks (say, less than 10 trucks per day) in spring thawing season without causing a significant damage and triggering a need for immediate intervention. The AASHTO 1993 Design Guide recommends 50,000 ESALs as a practical minimum design traffic loads for flexible and rigid pavement structures on low volume roads. Considering past practices and experience, and the AASHTO 1993 Design Guide recommendation, the following strategies are recommended for the design of a non-spring weight restricted highway:

i) If adequate budget is available, continue to provide design using a minimum of 25 trucks per day on the design lane (two-way 50 trucks per day). This pavement structure is expected to last longer if the actual spring damage is low.

ii) If the budget is inadequate, provide a design using a minimum ESALs of 50,000 or 10 trucks per day on the design lane, whichever provides the higher design traffic loads. Monitor the pavement for any potential or experienced damage and service condition in early life, especially during the spring thawing season. If the potential or experienced pavement damage and surface condition are major concerns, place an overlay or apply spring weight restrictions.

A layered design analysis is not required when designing pavement for less than 300,000 ESALs. However, in no case should the granular (total of base and subbase) thickness be less than 300 mm and AC layer thickness be less than 80 mm for a non-spring weight restricted road constructed on subgrade having a summer resilient modulus of ≤50 MPa.

6.17 Design for a Spring Weight Restricted Highway

Generally, Manitoba attempts to construct non-spring weight restricted highways/roads to support economic prosperity of industries and businesses, reduce burden on consumers, reduce fuel consumption and thereby the carbon footprint and improve highway safety and for sustainable uses of the natural resources. Therefore, a design for a spring weight restricted highway with reduced axle weights during the spring thawing season should be avoided, if possible. However, there could be cases where the construction of a full structure for a non-spring weight restricted highway section is not feasible due to budget constraints or not value added because the adjoining sections are spring weight restricted and there is no immediate plan to remove spring weight restrictions from those adjoining sections.

To provide the design for a spring weight restricted pavement structure, the calculated total structural number should be reduced to 70% before calculating the thickness of pavement layers. A layered design analysis is not required in this case. The spring weight restricted pavements should also be double chip seal surfaced to avoid break-ups during spring as well as other wet weather periods. If AC surfacing option is chosen, its thickness should be a minimum of 50 mm for a design ESALs of <150,000 and 60 mm for a design ESALs of 150,000 to <300,000. The total granular (base and subbase) thickness should be a minimum of 300 mm on subgrade having a summer resilient modulus of <35 MPa. Post construction FWD deflection testing should be conducted to determine the applicable spring restriction weight levels (Level 1 or Level 2) as per Manitoba’s Spring Road Restrictions policy.

6.18 Minimum AC Thickness Prior to Seasonal Shutdown

To ensure a long-term performance of AC pavement as designed, it is desirable that all AC layers/lifts be placed within a single construction season. If that is not possible for a project due to unavoidable circumstance(s), the construction project team should ensure that the Contractor complete certain minimum AC before seasonal shutdown of construction and opening the highway to traffic. The pavement designer should provide the recommendation for the minimum thickness of AC which is required to support the traffic loading during the shutdown period without causing any distresses in the partially completed AC pavement. The procedure outlined below can be used to determine the minimum AC thickness requirements:

1) Based on Subgrade Modulus:

   i) Design the pavement structure i.e., determine the design SN required for three (3) months service life (SN_{dgn-3M}) using spring modulus of subgrade and increasing the TEF by five times.

   ii) Calculate the SN of the required minimum AC thickness ($SN_{min-bit}$) by subtracting the SN of subbase and base layers (because full depth subbase and base layers are in place before the AC paving) from the SN_{dgn-3M}.

   iii) Calculate the required AC thickness for this $SN_{min-bit}$ using the appropriate structural layer coefficient of AC mixes.

2) Based on Granular Base Layer Modulus:

   i. Determine the required SN of surface layer for three (3) months service life (spring thawing period plus freeze-thaw cycles during November to March) (SN_{1-3M}) using the spring modulus of base layer and increasing the TEF by five times.

   ii. Calculate the required AC thickness for this SN_{1-3M} using the appropriate structural layer coefficient of AC mixes.

3) The required minimum AC thickness is the maximum from above two design scenarios. However, the thickness of AC layer should in no case be less than 80 mm prior to seasonal shutdown and opening to the traffic.

6.19 AC Thickness for Paved Shoulders

The thickness of base/subbase layer(s) on a shoulder should match with the respective thickness of base/subbase layer(s) on the adjacent main lane. Guideline for the selection of minimum thickness of paved (AC) shoulder is presented in Table 6.0.16. GBC-I, GBC-II or GBC-M should be used to fill the thickness discrepancy between AC paved shoulder and the adjacent main lane, where applicable. GBC-S should be used as the surface of the unpaved portion of the shoulders including gravel shoulder rounding.

Table 6.0.16: Minimum AC Thickness on Paved Shoulders

Paved Shoulder Width	Design ESALs	Travel Lane AC Thickness	Thickness of AC in Shoulders (Minimum)
≤1.0 m	All	All	Same as the adjacent main lanes
>1.0 m	All	<120 mm	Same as the adjacent main lanes
	<3.0 million	>120 mm	90 mm
	≥ 3.0 to <30.0 million		100 mm
	≥30.0 million		110 mm

Chapter 7: DESIGN FOR REHABILITATION AND PARTIAL DEPTH RECONSTRUCTION OF FLEXIBLE AND SEMI-FLEXIBLE PAVEMENTS

7.1 Design Inputs

The inputs for pavement design using the AASHTO 1993 Design Guide approach for rehabilitation and partial depth reconstruction projects are:

i) Existing pavement load carrying capacity
ii) Overlay design life and ESALs
iii) Subgrade stiffness
iv) Subgrade soils frost heave potential
v) Pavement serviceability
vi) Design reliability
vii) Overall standard deviation
viii) Drainage and environmental conditions
ix) Overlay material properties

7.2 Existing Pavement Load Carrying Capacity

The load carrying capacity of an existing pavement can be measured in terms of rebound deflection using Benkelman Beam or surface deflection basin using the FWD and the calculated effective structural number (SN_{eff}). However, the selection of appropriate treatment(s) of the existing paved surface before the placement of overlaying layer(s) is critical for cost-effective rehabilitation or partial depth reconstruction measure. The selection of treatment option and net structural capacity of an existing pavement, after any treatment of the existing surface (and base) layer(s), will depend on the thickness and condition of existing pavement materials, condition and strength of in-situ subgrade and the condition of existing pavement surface in terms of type and severity of observed distresses. Therefore, careful assessment of existing pavement is a key element for rehabilitation and partial depth reconstruction designs.

7.2.1 Assessment of Existing Pavement

As indicated earlier, pavement rehabilitation and partial depth reconstruction designs will require proper assessment of existing pavement and layer materials conditions. The department's highway inventory and pavement condition as well as maintenance databases should be thoroughly reviewed and analysed to assess pavement construction history, age, distress types, severity and trends, maintenance history, past rehabilitation as well as preservation treatments timing and their performance, and the pavement management system (PMS) outcomes in terms of recommended rehabilitation/reconstruction treatments. The PMS recommended outcomes should be verified through field investigation before providing the final design for use in the actual construction work.

Coring should be done in the bound surface layer (e.g., AC, AST, road mix and sand asphalt) when conducting the soil survey on an existing pavement structure. The thickness and type of each layer material of the existing pavement structures should be determined for all rehabilitation or partial depth reconstruction projects, in addition to the required soil survey. Pavement surface layer thickness (average of three measurements along the side of each core) at each core location should be reported to the nearest 0.001 m (1.0 mm). The depths of base and subbase layers at each core/borehole location should be reported to the nearest 0.01 m (10 mm) or less. Samples from each granular material (base and subbase) type should be collected and tested in the laboratory for moisture content, gradation, plasticity and classification in accordance with the department's Engineering Standard ENG- PG001 "*Soil Survey for Design and Assessment of Highway Pavements and Embankments*". Each material properties should be compared with the current and past specifications of granular base and subbase to assign appropriate structural value (structural layer coefficient or laboratory equivalent resilient modulus, as applicable) to the layer material.

All asphalt cores, taken as part of the site investigation, should be visually examined for the evidence of stripping, aging or layer delamination. If evidence of stripping or layer delamination is observed, a photograph should be taken with reference to the core location. Additional cores should be taken at randomly selected crack locations, in consultation with the Pavement Designer. The condition of these cores including crack type, crack width (at surface, mid-depth and bottom of each core) and the direction (top down or bottom up) of crack progression should be recorded, and photos should be taken.

The general condition of existing paved surface including the observed distresses should be recorded and possible reasons should be identified. Photographs of the existing pavement surface, shoulders and roadsides including ditches should also be taken. The depth from pavement surface to the prairie ground surface and bottom of the adjacent ditches should be measured. Rut depth (mm) and cross-fall measurements (%) should be taken on main lanes and shoulders where core/boreholes are drilled. Rut depths should be measured using a 1.2 m long straight edge, on both wheel paths, and the cross-falls should be measured using a 3.0 m long straight edge.

Any areas with localized unusual distresses or failures should be thoroughly investigated to determine the causes in consultation with the Pavement Designer. The required information related to frost heave, settlement and swelling issues should be collected from the regional maintenance team.

All the above information should be used to determine the feasible alternative treatments of the existing pavement and the effective structural number, where required.

7.2.2 Ground Penetrating Radar Data

The core and boreholes for pavement investigation are usually done at a specified interval depending on the project type and available resources. The specified frequencies for core and boreholes cannot determine the type and thickness of each layer at every point along a highway alignment, whether existing or proposed. The Ground Penetrating Radar (GPR), which uses dielectric constants to identify different type of materials, can aid in the determination of variation in each layer material type and thickness. The accuracy of layer thickness data from GPR depends on the quality of thickness mapping program or approaches used in the GPR system. The layer thickness data from a GPR may not be accurate enough for use in the pavement structural capacity assessment. However, this data can provide valuable information regarding unusual type of localized material and thickness for further exploration or investigation through coring and borehole drilling in concerned areas. The accuracy of the GPR scanned data can also be improved through the selection of appropriate equipment for different applications and proper calibration of the selected equipment.

7.2.3 Pavement Surface Deflection Data

Over the past several decades, Manitoba had been using the BBR deflection for determining load carrying capacity and the overlay requirements of existing AC, road mix and AST surfaced pavement structures. The BBR deflection was measured using a 3.65 m long beam with a mounted deflection gauge to measure the vertical rebound of a pavement. A 2-axles (steer and single axles) straight truck was loaded with gravel/stone material to impose an 80 kN static load on the single axle unit, which corresponds to one standard axle load repetition (one ESAL) on a pavement surface. The beam was placed between dual wheels of the single axle assembly in the outer wheel path side of the axle. The deflection gauge recorded the rebound of the pavement after the truck is driven away. The measured BBR deflection value from a pavement section provided the load carrying of that pavement in terms of allowable ESALs prior to any structural enhancement. Manitoba had an extensive database of BBR deflection for the entire paved surface network. Since no data has been collected since 2008 and Manitoba switched to the collection and use of FWD deflection data, the BBR data has been archived.

Manitoba has been collecting the falling weight deflectometer (FWD) deflection data utilizing external service providers since the beginning of the 1990's from different research projects and some selected highway sections at network level. Manitoba acquired its first FWD equipment in 2008 and completed two rounds of data collection from almost the entire paved surface network by the end of 2020. Project and research levels data are being collected as needed. All new rehabilitation/reconstruction designs and analysis of existing pavements should be based on FWD deflection data.

FWD Data

FWD equipment applies a dynamic impulse load on the pavement surface that simulates a moving wheel load from heavy (commercial) vehicles. The FWD is now the foremost device for structural assessment of pavements at network, project and research levels. It measures the actual deflections of pavements as opposed to the rebound deflections measured by the Benkelman Beam. The measured FWD deflections are more accurate and repeatable than BBR. The FWD is also equipped with a series of geophones, which can be positioned at different radial distances on both sides of the load plate, as opposed to a single deflection gauge with the Benkelman Beam device. This allows for the measurement of pavement deflections at the centre as well as away from the centre of the FWD load plate for different purposes including deflection basin analysis, stress-strain analysis and other parameters. Several models and software are available to estimate the pavement layers and subgrade moduli and the structural capacity of the existing pavements.

The FWD testing for different applications should be conducted following the latest version of the Engineering Standard "*ENG- P008: Deflection Testing Using the DYNATEST® Falling Weight Deflectometer*". The deflection data should be normalized to standard load and temperature as described in the above specified standard before any structural assessment of an existing pavement including the determination of load carrying capacity in terms of effective structural number and the subgrade modulus. The deflection basin data can be used for the determination of moduli of pavement layers and subgrade for use in design approaches other than AASHTO 1993 (e.g., Pavement ME Design), research and analysis of pavements.

The project level FWD deflection data should not be more than three years old for a pavement design that will be used for actual construction purposes. An older FWD data can be used for preliminary pavement design and budget estimate purposes.

7.2.4 Determination of Effective Structural Number

AASHTO 1993 Pavement Design Guide has provided three approaches to determine the load carrying capacity in terms of effective structural number of an existing pavement. These approaches are: i) remaining life; ii) visual survey and estimate or measure structural layer coefficients; and iii) non-destructive deflection test. In the "*remaining life approach*", an existing pavement's load carrying capacity is estimated based on the traffic loads to failures, traffic loads that the pavement has already experienced to date, original pavement structural capacity and the structural capacity after to date traffic exposure. Obtaining or estimating this information is a difficult task, if not impractical, and therefore this approach is considered unsuitable for Manitoba.

In the second i.e., "*visual survey and estimate or measure structural layer coefficients approach*", an existing pavement's effective SN (SN_{eff}) is calculated based on the thickness of each layer and its structural layer coefficient. AASHTO 1993 design guide has provided guideline for subjective estimation of structural layer coefficient of in-situ AC layer based on the extent and severity of cracks. Laboratory testing for determining the appropriate layer coefficient is resource intensive, especially for the surface layer. The subjective estimation for layer coefficient of each layer material is an option, but it may not be very accurate or dependable for final/detailed design and construction purposes. This option can only be used to provide the preliminary designs for pavement rehabilitation and partial depth reconstruction at functional design stage or rough estimate of the required project budget when the project level FWD deflection and coring data are unavailable.

In the third approach, which is the "*non-destructive deflection test approach*", an existing pavement's structural capacity is estimated using the backcalculated effective modulus and thickness of the entire pavement structure. The effective modulus of a pavement structure is estimated using backcalculated layer modulus of subgrade from FWD deflection data at 20 °C, FWD central deflection value at 20 °C, radius of FWD load plate, applied pressure on the FWD load plate and the total thickness of the pavement structure. This approach is considered more accurate than other two approaches stated above as the measured data, using a well-accepted technology that accounts for the condition of the existing pavement, are used. Therefore, Manitoba has adopted this third approach to determine the effective SN (SN_{eff}) of existing flexible and semi-flexible pavements, especially for the final/detailed design and construction purposes. The "*visual survey and estimate structural layer coefficient approach*" should be used as a supplemental approach to the "*non-destructive deflection test approach*" to estimate the structural layer coefficient(s) of the existing pavement material(s) that will be removed as part of pre-overlay treatment of an existing pavement structure.

Visual Survey and Estimate Structural Layer Coefficients Approach

For the preliminary design, the structural layer coefficients of an existing pavement layer materials that will remain in place can be estimated based on the surface condition, assessment of cores and layer material types. The structural layer coefficients of typically used materials and typical conditions are provided in Table 7.0.1 as a guideline. An appropriate layer coefficient should be assigned to a layer material that falls outside the list presented in Table 7.0.1 such as soft AC surface that experiences unexpected significant amount of rutting or any bleeding issues. An AC surface can be considered soft the AC layer alone experiences greater than 6.0 mm of rutting within the first seven (7) years and/or greater than 12 mm of layer rutting within the first 20 years the following the placement of that AC layer.

After knowing the thickness of each layer material that will remain in place without any stabilization or treatment application, the effective SN (SN_{eff}) can be calculated using the following formula:

$$SN_{eff} = \sum D_{ie} a_{ie} \tag{7.1}$$

where,

D_{ie} = thickness of layer i (1, 2, 3, 4………..) of existing pavement that will remain in place without any treatment

a_{ie} = structural layer coefficient value of layer i (1, 2, 3, 4………..) of the existing pavement that will remain in place without any stabilization or other treatment

Table 7.0.1: Structural Layer Coefficients of Layer Materials to Remain in Place

Layer Material	Visual Observation of Cores and Layer Materials	Structural Layer Coefficient
AC	Very Good: No surface cracks; asphalt matrix is well bonded/strong and no sign of aging or moisture related damage	Good quality aggregates: 0.40 Fair quality aggregates: 0.35 (Note 1)
	Good: Few surface cracks; asphalt matrix is well bonded but slightly aged; slight moisture related damage	Good quality aggregates: 0.35 Fair quality aggregates: 0.30 (Note 1)
	Fair: Frequent surface cracks; asphalt matrix is well bonded but moderately aged; moderate moisture related damage	0.25
	Poor: Abundant surface cracks; asphalt matrix is fairly bonded and cores are still intact; substantially aged and substantial moisture related damage	0.20 (Note 2)
	Very Poor: Surface cracks are throughout/ extensive; asphalt matrix is brittle and no intact cores; severe moisture related damage	Pulverize and relay or remove and replace (or recycle)
AST, Asphalt Bound Road Mix or Maintenance Mix	Fair to good condition	Thickness ≥40 mm: 0.15 Thickness <40 mm: Ignore
	Poor to very poor condition with extensive cracks	Mill/remove or mill/pulverize and relay
Sand Asphalt	Well bonded mix	0.12
GBC- I, GBC- II, GBC- M and GBC- S	In dense condition with a low moisture content (≤ optimum moisture)	90% of values specified in Table 6.0.10 (Chapter 6)
GSB- C, GSB- F, CR-M50, CR- M100, CR-M125	In dense condition with a low moisture content (≤ optimum moisture)	90% of values specified in Table 6.0.10 (Chapter 6
Well Graded Granular Base	In dense condition with a low moisture content (≤ optimum moisture)	% Fines ≤9.0: 0.12 % Fines >9.0 to 12: 0.11 % Fines >12 to 15: 0.10
Well Graded Granular Subbase	In dense condition with a low moisture content (≤ optimum moisture)	% Fines ≤9.0: 0.10 % Fines 9.0 to 12: 0.09 % Fines >12 to 15: 0.08
Crushed Rock	Graded and clean	0.14
Cement Stabilized Base	Good condition (Note 3)	0.16
	Poor condition (Note 3)	0.12
Cement or Lime Stabilized Subgrade	Good condition (Note 3)	0.06
	Poor condition (Note 3)	Ignore

Note 1: Primarily in South-Western Manitoba, but could be present in other areas of the province

Note 2: Consider deep (≥50 mm) milling or pulverizing and relaying of the existing AC layer

Note 3: Based on visual observation and/or resistance to coring/drilling

An existing pavement layer or any part of it that will be milled and removed (hauled away), and milled and re-laid with or without any stabilization treatment such as mill and relay, CIR, FDR and pulverization should not be included in the SN_{eff} calculation. The layer or part of it that will be milled and re-laid, stabilized or recycled and re-laid, and pulverized and re-laid on the roadway will be considered as a new overlay layer.

<u>Non-destructive Deflection Test Approach</u>

In this approach, the effective SN (SN_{eff}) at each FWD deflection test point of an existing pavement is calculated using the following formula (Equation 7.2):

$$SN_{eff} = 0.0045 * D * \sqrt[3]{E_p} \tag{7.2}$$

where,

D = total thickness of pavement layers (surface, base and subbase) above the subgrade of an existing pavement, inches

E_p = effective modulus of all pavement layers above the subgrade, psi

The total thickness (D) of pavement can be taken as the average thickness from all cores and boreholes or test pits within each subsection of a project area with uniform central deflection values and applied to each FWD deflection test point within that subsection, especially when the FWD deflection test points do not match with core/bore or test pit points. The E_p should be determined using the following equation (Equation 7.3) from the AASHTO 1993 design guide:

$$d_o = 1.5 * p * a * \left\{ \frac{1}{M_R * \sqrt{1+\left(\frac{D}{a} * \sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{\left[1 - \frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]}{E_p} \right\} \tag{7.3}$$

where,

d_0 = deflection measured at the centre of the FWD load plate (corrected to the standard temperature of 20°C and stress of 566 kPa), inches

p = standard stress on the FWD load plate, psi

a = radius of the FWD load plate, inches

D = total thickness of pavement layers above the subgrade, inches
M_R = backcalculated resilient (layer) modulus of subgrade (uncorrected to equivalent laboratory modulus value), psi
E_p = effective modulus of all pavement layers above the subgrade, psi

The following equation (Equation 7.4) from the AASHTO 1993 design guide should be used to backcalculate the resilient (layer) modulus of subgrade soils at each FWD test point:

$$M_R(psi) = \frac{0.24 * P}{d_r * r} \quad (7.4)$$

where,

M_R = backcalculated resilient (layer) modulus of subgrade (uncorrected), psi
P = applied load, lbs
d_r = measured deflection at radial distance *r* from the centre of the plate (corrected to the standard temperature of 20°C and stress of 566 kPa), inches;
r = radial distance from the centre of the FWD load plate at which the deflection is measured (i.e., distance to each geophone position), inches

The representative backcalculated resilient modulus at each FWD test point should be taken for the geophone position that corresponds to a minimal radial distance from the centre of the FWD load plate (professional judgement should also be applied in the selection of representative geophone location). The minimum radial distance should be determined using the following equation from AASHTO 1993 design guide:

$$r \geq 0.7 * a_e \quad (7.5)$$

where,

$$a_e = \sqrt{\left[a^2 + \left(D * \sqrt[3]{\frac{E_p}{M_R}}\right)^2\right]}$$

r = radial distance at which the deflection is measured, inches
a_e = radius of the stress bulb at the subgrade-pavement interface, inches
a = radius of the FWD load plate, inches
D = total thickness of pavement layers above the subgrade, inches
E_p = effective modulus of all pavement layers above the subgrade, psi
M_R = backcalculated resilient (layer) modulus of subgrade (uncorrected to

equivalent laboratory modulus value), psi

The representative effective SN for a highway/road section with uniform strength (similar central deflection values) should be taken as the average of all effective SN values corresponding to all the FWD deflection test points within that section. Any isolated high and unexpected low effective SN values (which are considered outliers) should be screened out so that the coefficient of variation (CoV) of a set of effective SN values, representing a highway/road section, do not exceed the limit calculated using Equation 7.6.

$$CoV = 100 - R \tag{7.6}$$

where,

CoV = coefficient of variation (standard deviation divided by average), %

R = selected design reliability, %

An isolated area with a very low effective SN, which is screened out, should be considered localized weak area. Additional overlay layer(s) should be placed on such isolated areas, wherever feasible.

The representative effective SN calculated using the above specified equations (Equations 7.2 to 7.6) reflects the overall structural number of a uniform section of an existing pavement structure. If the existing pavement treatment will include mill and remove or mill and relay (with or without any stabilization), the SN loss due to any such treatment(s) should be subtracted from the representative effective SN to determine net representative effective SN of the remaining unaltered pavement layers. Any re-laid material, with and without stabilization, should be considered part of the overlay. The net representative effective SN can be calculated using the following equation (Equation 7.7):

$$SN_{eff\_net} = SN_{eff\_rep} - S(D_{ir} * a_{ir}) \tag{7.7}$$

where,

SN_{eff_net} = net representative effective SN after milling of any existing pavement layer material(s) in a uniform pavement section

SN_{eff_rep} = representative effective SN before milling of any existing pavement layer material(s) in a unform pavement section

D_{ir} = milling thickness of layer i material

a_{ir} = structural layer coefficient of material i that will be milled

The structural layer coefficients of existing pavement layer materials that will be milled and removed or milled and re-laid on the roadway with or with any stabilization treatment can also be estimated based on the assessment of surface condition, cores and the layer material type and properties. The guideline presented in Table 7.0.2 can be used for selecting the structural layer coefficients of typically used materials. An appropriate layer coefficient should be assigned for a layer material that falls outside the list presented in Table 7.0.2 such as soft AC surface that experiences unexpected significant rutting or any bleeding issues.

7.3 Overlay Design Life and ESALs

For rehabilitation or partial depth reconstruction projects, AC and AST pavements should be designed to provide 20 years initial service life at a preselected minimum service quality without any structural enhancement or AC resurfacing. A shorter design service life can be considered for special cases, e.g., for roadway section with frost heave and/or swelling issues and when the roadway section in question will be removed and relocated or reconstructed within next 10 years. The design traffic loads i.e., the accumulative standard load repetitions or ESALs over the selected design service life should be calculated using Equation 4.1 with the appropriate TEF as outlined in Chapter 4. All routes classified as trade or commerce in department's strategic classification system should be designed to handle RTAC loads regardless of traffic volume and functional classification.

7.4 Subgrade Soil Stiffness

For overlay designs for pavement rehabilitation and partial depth reconstruction projects, the subgrade resilient modulus should be determined through backcalculation from FWD deflection basin data. The process is outlined in Section 5.2.4 (Chapter 5). If FWD deflection data is unavailable, a suitable alternative method from those discussed in Chapter 5 can be chosen to estimate the resilient modulus of subgrade soils. However, the estimated resilient modulus based on soil type and contents can only be used for preliminary design.

Table 7.0.2: Structural Layer Coefficients of Layer Materials to be Removed

Layer Material	Visual Observation of Cores and Layer Materials	Structural Layer Coefficient
AC	Very Good: No surface cracks; asphalt matrix is well bonded/strong and no sign of aging or moisture related damage	Overlay, if required (no milling)
	Good: Few surface cracks; asphalt matrix is well bonded but slightly aged; slight moisture related damage	Bit. B: 0.40 SP 12.5: 0.42 SP19.0: 0.44
	Fair: Frequent surface cracks; asphalt matrix is well bonded but moderately aged; moderate moisture related damage	0.35
	Poor: Abundant surface cracks; asphalt matrix is fairly bonded and cores are still intact; substantially aged and substantial moisture related damage	0.30
	Very Poor: Surface cracks are throughout/ extensive; asphalt matrix is brittle and no intact cores; severe moisture related damage	Pulverize and relay or remove and replace (or recycle)
AST, Asphalt Bound Road Mix or Maintenance Mix	Fair to good condition	0.20
	Poor to very poor condition with extensive cracks	0.15
Sand Asphalt	Well bonded mix	0.14
GBC- I, GBC- II, GBC- M and GBC- S	In dense condition with a low moisture content (≤ optimum moisture)	Refer to Table 6.0.10 (Chapter 6)
Well Graded Granular A Base	In dense condition with a low moisture content (≤ optimum moisture)	% Fines ≤9.0: 0.14 % Fines >9.0 to 12: 0.13 % Fines >12 to 15: 0.12
Cement Stabilized Base	In dense condition with a low moisture content (≤ optimum moisture)	0.18
	Graded and Clean	0.14

**Based visual observation or resistance to coring/drilling*

For economical pavement overlay structures, the project length on a particular highway section can be subdivided into smaller subsections based on the uniformity in FWD central deflection values. If no FWD deflection data is available, existing pavement layer thickness and subgrade type can be used to divide the project area into subsections, if required. The ease and effectiveness of construction activities should also be considered in sub-sectioning process, where applies. Generally, a subsection length should not be less than 2.0 km, unless the total length of the highway section under construction is less than 2.0 km in length. Once the highway section in a particular project is divided into subsections, the representative and the effective resilient modulus for each subsection should be determined using the procedures described in Chapter 5.

7.5 Subgrade Soils Frost Heave Potential

Refer to Chapter 6, Section 6.4.

7.6 Design Serviceability Loss Due to Frost Heave

Refer to Chapter 6, Section 6.5.

7.7 Pavement Serviceability

Refer to Chapter 6, Section 6.6 for the selection of initial and terminal serviceability index values. Both of intact AC, depending on the surface as well as layer conditions and thickness, and asphalt cement or emulsion stabilized/treated reclaimed/recycled asphalt pavement should be added to the number of new overlay AC lift(s) when selecting the Initial Serviceability Index values. Guideline is provided in Table 7.0.3 for considering the number of lifts for the existing AC pavement including the intact (untreated) AC and treated/stabilized reclaimed AC layers. Pulverized asphalt, without or with emulsion added to aid the placement and traffic movement, should not be considered a bound layer and it should not be included in the count for the number of lifts for selecting the initial serviceability index values.

7.8 Design Reliability

Refer to Chapter 6, Section 6.7.

7.9 Overall Standard Deviation

Refer to Chapter 6, Section 6.8.

Table 7.0.3: Number of Lifts Considered for the Existing AC Layers

Existing Pavement Surface and Treatment	Condition	Number of Lift(s)
AST, Road Mix and Maintenance Patch	N/A	0
Un-milled AC (Straight Overlay Option)	IRI ≤1.5 m/km and/or rut depth ≤ 6 mm	1
	IRI >1.5 m/km and/or rut depth ≥7 mm	0
Partially (25 to 50 mm) Milled AC (Mill and Overlay Option)	Thickness of remaining intact AC below the new overlay AC layer = 50 to 85 mm	1
	Thickness of remaining intact AC below the new overlay AC layer >85 mm	2
Full Depth Removal of AC	N/A	0
Pulverized Asphalt, New RAP or New Granular	N/A	0
CIR and CCPR	Thickness of remaining intact AC below the new overlay AC layer = 50 to 85 mm	2 (including the CIR/CCPR layer)
	Thickness of remaining intact AC below the new overlay AC layer >85 mm	3 (including the CIR/CCPR layer)
FDR	N/A	1

7.12 Overlay Structure for Rehabilitation and Partial Depth Reconstruction

The required overlay on a uniform existing pavement section is determined based on the total (design) structural number and the net effective structural number of that section. The calculation process for determining the net effective structural number has been described earlier in this Chapter. The process for determining the total (design) structural number is the same as that presented in Chapter 6 for new construction or full depth reconstruction. As stated in Chapter 6, the total (design) SN required to withstand traffic load repetitions over the design service life for a given subgrade stiffness, pavement serviceability and reliability is calculated using Equation 6.10. Refer to Chapter 6 for details of the equation and tool/process to solve it for determining the design SN. The SN of the required overlay can be calculated using the following equation (Equation 7.8):

$$SN_{Ol} = SN_{dgn} - SN_{eff\_net} \tag{7.8}$$

where,

SN_{Ol} = *structural number of overlays*

SN_{dgn} = *design (total) structural number (same as a new pavement)*

SN_{eff_net} = net representative effective SN after milling of any existing pavement material(s) in a road section

7.12.1 Determination of Overlay Thicknesses

The overlay SN (SN_{Ol}) can be converted into thicknesses of different layer materials using the effective layer coefficient (a) values of the material(s) that will be used in the actual rehabilitation or partial depth reconstruction of pavements. The following equation (Equation 7.9) can be used when using the effective layer coefficients of overlay material(s):

$$SN_{Ol} = \sum D_{i\_Ol} * a_{i\_Ol} \tag{7.9}$$

where,

D_{i_Ol} = net thickness of overlay layer i (1, 2, 3, 4………..)

a_{i_Ol} = structural layer coefficient value of overlay layer i (1, 2, 3, 4………..)

The structural layer coefficients of overlay granular base and AC layers can be selected from Tables 6.0.9 and 6.0.10 (Chapter 6), respectively. The structural layer coefficients of reclaimed, recycled, treated and re-laid materials can be selected from Table 7.0.4.

When determining the required thickness of AC layer(s), which is required to meet the SN_{Ol}, to be placed on the top of an un-milled existing AC (i.e., for straight overlay without any milling), pulverized asphalt and granular base, the designer should consider about 12.5 mm AC loss (no considerable structural contribution) at the bottom of new overlays for levelling. This extra thickness for levelling should be added to the calculated overlay AC thickness when recommending the overlay thickness in the Pavement Structure and Surfacing Design Memo (PSSDM) or pavement design report.

Table 7.0.4: Structural Layer Coefficients of Reclaimed, Recycled and Treated Materials

Material	*Effective Layer Coefficient
Good Quality Pulverized Asphalt and Processed RAP	0.14
Milled or Pulverized AST/Road Mix/Maintenance Mix	0.12
CIR and CCPR Asphalt of Bit. B/Bit. C Mixes	0.25
CIR and CCPR Asphalt of SuperPave AC Mixes	0.30
FDR Asphalt of Bit. B/Bit. C Mixes	0.20
FDR Asphalt of SuperPave AC Mixes	0.25
Mixture of Pulverized Asphalt and In-situ Granular Base	Determine based on the thickness ratio of pulverized asphalt and base layers

** Change based on the quality of materials/aggregates*

7.12.2 Minimum Thickness of Asphalt Concrete Layer

Follow the steps outlined below to determine the minimum thickness of asphalt overlays:

1) Determine the structural number (SN_1) of the required total minimum AC layer(s) using the resilient modulus value of unbound layer material, such as pulverized asphalt, RAP or granular base, which exists right below the bottom layer of AC (existing or new). Refer to Sections 6.11.2 and 6.12 (Chapter 6) for the procedures to determine the SN_1.
2) Use the appropriate structural layer coefficients (refer to Tables 7.0.1 and 7.0.4) of the existing remaining plus CIR (or CCPR) asphalt layers or FDR asphalt layer, as applicable, to calculate their respective structural number(s).
3) Subtract the calculated structural number(s) of the existing remaining plus CIR (or CCPR) asphalt layers or FDR asphalt layer, as applicable, from SN_1 to determine the structural number of the required minimum new AC overlay layer.
4) Convert the SN of the required minimum new AC to its layer thickness using appropriate structural layer coefficient of the new AC material to be used.

7.13 Overlay Design Examples

Example 1: Visual Survey and Estimate Structural Layer Coefficients Approach (Preliminary Design)

<u>Highway Information</u>

a) Highway: A provincial two-lane undivided expressway in Capital Region (climate zone 1A)
b) Existing pavement (Highway Inventory data): 100 mm AC (poor pavement condition with IRI >2.5 m/km) and 150 mm granular base
c) Highway loading classification: RTAC
d) Traffic volume: AADT of 4,500 with 160 trucks/day (2-way) and 0.7% annual growth rate
e) Overlay design service life: 20 years
f) Subgrade type: Sandy clay
g) Subgrade soils frost heave potential: Negligible
h) Drainage and environmental conditions (highway context): Rural
i) Pavement overlay materials: Bit. B surface and GBC- I base (if required)

<u>Overlay Design Parameters</u>

Design traffic loads (ESALs) with a DLF of 0.5 and TEF of 1.055 = 659,000
Subgrade effective resilient modulus (based on network level FWD deflection data) = 30.7 MPa
Design reliability = 90%
Initial serviceability index = 4.3 for three AC lifts and 4.4 for four lifts
Terminal serviceability index = 2.5
Serviceability loss due to traffic (no loss due to frost) for three lifts = 4.3 - 2.5 = 1.8
Serviceability loss due to traffic (no loss due to frost) for three lifts = 4.4 - 2.5 = 1.9
Overall standard deviation = 0.45
Layer coefficients of overlays: Bit. B = 0.40, pulverized asphalt = 0.14 and GBC- I = 0.146
Layer coefficient of existing layer materials to remain in place: AC = 0.20, granular base = 0.10

<u>Effective SN, Design SN and Overlay Layer Thickness</u>

The calculated total (design) SN_{dgn} (three lifts AC) = 99.0 mm
The calculated total (design) SN_{dgn} (four lifts AC) = 97.8 mm

Option 1: Straight Overlay

SN_{eff} = 100*0.20 + 150*0.10 = 35.0 mm
SN_{OI} = 99.0 - 35.0 = 64.0 mm
Overlay AC thickness = 64.0/0.40 = 160 mm
Adding 10 mm for levelling, required overlay thickness is **170 mm thick Bit. B**

Option 2: Mill 35 mm and Overlay

SN_{eff} (remaining AC plus base) = (100 - 35) * 0.20 + 150*0.10 = 28.0 mm
SN_{OI} = 97.8 - 28.0 = 69.8 mm
Overlay AC thickness = 60.8/0.40 = 174.5 mm; say, **175 mm thick Bit. B**

Option 3: Pulverize and Overlay

Pulverize 100 mm AC, relay on main lanes and shoulder that will produce 65 mm pulverized asphalt on the main lanes and add GBC- I and Bit. B, as required.
SN_{eff} (granular base layer) = 150*0.10 = 15.0 mm
SN of pulverized asphalt = 65*0.14 = 9.1 mm
150 mm new GBC- I (placed over pulverized asphalt) = 150*0.146 = 21.9 mm
SN_{OI} AC = 99.0 – (15.0 + 9.1 + 21.9) = 53.0 mm
Overlay AC thickness = 53.0/0.40 = 133 mm Bit. B
Check whether the minimum AC thickness criteria is met. If not, reduce the GBC- I thickness to meet the minimum Bit. B thickness requirement.
Add 12.5 mm for levelling; say, **145 mm thick Bit. B** will be placed if 133 mm Bit. B meet the minimum requirement.

Example 2: Non-destructive Test Approach (Detailed Design)

Highway Information

a) Highway: A provincial two-lane undivided arterial highway (NHS - core route) in Central Region (climate zone 1A)
b) Existing pavement (coring/drilling data): 120 - 195 mm (avg. 145 mm) AC (fair condition with IRI >1.5 m/km, rut depth >6.0 mm) and 385 mm granular base/subbase (total thickness = 530 mm).
c) Highway loading classification: RTAC
d) Traffic volume: AADT of 1,650 with 500 trucks per day (2-way) and 2.4% annual growth rate

e) Overlay design service life: 20 years
f) Subgrade type: Sandy clay
g) Subgrade soils frost heave potential: Negligible
h) Drainage and environmental conditions (highway context): Rural
i) Pavement overlay materials: Bit. B

Overlay Design Parameters

Design traffic loads (ESALs) with a DLF of 0.5 and TEF of 1.783 = 4,100,000
Subgrade effective resilient modulus (based on FWD data) = 36.7 MPa (average for the section with M_R calculated for each FWD deflection test point)
Design reliability = 90%
Initial serviceability index = 4.2 or 4.4, depending on the surface condition and number of AC lifts
Terminal serviceability index = 2.5
Serviceability loss due to traffic (no loss due to frost) = 4.2 - 2.5 = 1.7, or 4.4 - 2.5 = 1.9
Overall standard deviation = 0.45
Layer coefficients of overlays: Bit. B = 0.40, pulverized asphalt = 0.14, CIR asphalt = 0.25
Layer coefficient of existing layer materials to be milled: AC = 0.35, granular base =0.12

Effective SN, Design SN and Overlay Layer Thickness

Effective SN (SN_{eff}) of the existing pavement = 94.5 mm (average for the section with SN_{eff} calculated for each FWD deflection test point)

Option 1: Straight Overlay

The calculated total (design) SN_{dgn} = 124.3 mm (with an initial serviceability index of 4.2 for two lifts of AC overlay on un-milled) surface
SN_{OI} = 124.3 – 94.5 = 29.8 mm
Overlay AC thickness = 29.8/0.40 = 74.5 mm
Adding 12.5 mm for levelling, required overlay thickness is 87 mm; say, **90 mm thick Bit. B**

Option 2: Mill 35 mm and Overlay

SN_{eff_net} = 94.5 - 35*0.35 = 82.2 mm
The calculated total (design) SN_{dgn} = 120.5 mm (with an initial serviceability index of 4.4 for 85 mm thick AC remain in place after milling and two lifts of new AC overlay)
SN_{OI} = 120.5 – 82.2 = 38.3 mm

Overlay AC thickness = 38.3/0.40 = 96 mm; say, **100 mm thick Bit. B**

Option 3: Pulverize and Overlay

Pulverize 200 mm (145 mm AC plus 55 mm granular base), relay on main lanes and shoulders that will produce a 130 mm thick layer of pulverized asphalt on the main lanes and then overlay with Bit. B, as required.
SN_{eff_net} = 94.5 - (145*0.35 + 55*0.12) = 37.2 mm
The calculated total (design) SN_{dgn} = 120.5 mm (with an initial serviceability index of 4.4 for four lifts of AC overlay over pulverized asphalt)
Structural layer coefficient of pulverized asphalt = (145*0.14 + 55*0.12)/200 = 0.135
SN of pulverized asphalt layer = 130*0.135 = 17.5 mm
SN_{Ol} of required AC overlay = 120.5 – (37.2 + 17.5) = 65.8 mm
Overlay AC thickness = 65.8/0.40 = 165 mm Bit. B
Check whether the minimum AC thickness criteria is met.
Adding 12.5 mm for levelling, total 177 mm; say, **180 mm thick Bit. B** will be placed if it meets the minimum requirement.

Option 4: Cold In-Place Recycle and Overlay

Cold In-place Recycle (CIR) the top 70 mm of the existing AC layer and relay on the asphalt paved areas to produce a 70 mm thick layer CIR asphalt on the main lanes.
SN_{eff_net} = 94.5 – 70*0.35 = 70.0 mm
The calculated total (design) SN_{dgn} = 120.5 mm (with an initial serviceability index of 4.4 for four lifts of AC with two lifts of new AC overlay on CIR asphalt layer, CIR asphalt lift and the remaining existing AC layer after milling)
SN of CIR asphalt layer = 70*0.25 = 17.5 mm
SN_{Ol} of new AC overlay layer = 120.5 – (70.0+17.5) = 33.0 mm
Overlay AC thickness = 33.0/0.40 = 83 mm; say, **85 mm thick Bit. B** will be placed.

7.14 Overlay Design with Frost Heave Management

The construction of a thick overlay is not likely to be an economically feasible option to mitigate frost heave related pavement performance issues. As such, in general, the overlay pavement thickness should not be increased as a frost heave mitigation measure. The following practices to manage the frost heave issues (which is similar to the design for new construction and full depth reconstruction), instead of costly treatment of an existing pavement and/or increase in overlay thickness to control of frost heave, are recommended in pavement design for rehabilitation and partial depth reconstruction projects:

i) Where practical, remove frost susceptible soils to a depth of 50% (70% for NHS Core and Intermodal routes) of the total frost depth measured with respect to the final pavement surface at isolated areas with severe and very severe frost heave issues. Add subbase/base and AC layers as determined through pavement design for full depth reconstruction (refer to Chapter 6). Ensure a minimum service life of 10 years including serviceability loss due to frost heave.

ii) Construct pavement overlay structure with pavement design for a reduced service life, which is developed through the following procedure:

1. Determine the required pavement structure (total SN) for 20 years performance period without considering the serviceability loss due to frost heave i.e., using 20 years design traffic loads, subgrade M_R, design reliability and overall (total) serviceability loss (ignoring the serviceability loss due to frost heave).
2. Assume the expected (trial) service life and the total thickness of pavement structure including the existing pavement structure (after any milling and removal) plus overlay(s) under both traffic loads and frost heave conditions. A lower performance period is expected for a greater serviceability loss (based on frost severity and probability) due to the frost heave.
3. Calculate the individual serviceability loss due to frost heave and traffic loads (refer to Chapter 6 for the procedure).
4. Calculate the required total SN for the expected (i.e., reduced) service life of the initial pavement for the serviceability loss due to traffic loads alone.
5. Repeat Steps #2 to #4 until the calculated SN due to traffic loads alone matches with the design SN determined in Step #1.
6. The service life (i.e., performance period) of the initial pavement structure should not be less than 10 years. If the calculated service life of the initial pavement structure in Step #5 is less than 10 years, determine the required SN for 10 years service life of the initial pavement structure under both traffic loads and frost heave conditions. This can be done through the following procedure:

a) Set the expected service life of the initial pavement structure as 10 years (or more, if desired)

b) Assume a higher total thickness of pavement structure than that was assumed in Step # 2.

c) Follow Steps # 3 and 4.

7. Confirm that the total thickness of all layers matches with the assumed total thickness in Step # 2 or # 6(b), as applicable.

8. Indicate the timing (year) of the expected overlay requirement in the Pavement Structure and Surfacing Design Memo (PSSDM) or pavement design report.

iii. Overlay or mill and overlay the pavement when the serviceability level falls below the desirable level.

7.15 Design Adjustment for Organics in Subgrade Soils

Refer to Section 6.15 (Chapter 6) for seasonal factors of subgrade modulus variation or the design SN adjustment, as applicable.

7.16 Minimum Pavement Structure for a Non-Spring Weight Restricted Highway

Refer to Section 6.16 (Chapter 6) for the design (total) SN calculation. Then follow the procedures for the effective SN (refer to Section 7.2.4) and overlay SN (refer to Section 7.12) calculation to determine the required overlay thickness (refer to Section 7.12.1).

7.17 Design for Spring Weight Restricted Highway

Refer to Section 6.17 (Chapter 6) for the design (total) SN calculation.

7.18 Minimum AC Thickness Prior to Seasonal Shutdown

The procedure outlined below can be used to determine the minimum AC overlay thickness requirements prior to seasonal shutdown:

1) Based on Subgrade Modulus:

i) Determine the design (total) structural number for three (3) months service life (SN_{dgn-3M}) using the spring modulus of subgrade and increasing the TEF by five times.

ii) Determine the total structural number of all layers that will be in place (SN_{exist}) prior to the placement of AC overlay. This will depend on the selected rehabilitation or partial depth reconstruction option such as:

a) Straight Overlay: SN of all existing layers of a pavement structure.

b) Mill and Overlay: SN of all existing layers of a pavement structure excluding milled AC, road mix or AST, as applicable. Add the SN of other layer material placed below the new AC overlay, if applies.

c) Pulverize and Overlay: SN of all existing layers of a pavement structure excluding milled AC plus the SN of pulverized asphalt and any additional CCPR asphalt and/or new granular base material, if placed over the pulverized asphalt.

d) CIR and Overlay: SN of all existing layers of a pavement structure excluding milled/reclaimed AC plus the SN of CIR asphalt.

e) FDR and Overlay: SN of all existing layers of a pavement structure excluding milled/reclaimed AC plus the SN of FDR asphalt.

iii) Calculate the structural number of the required minimum AC thickness ($SN_{min-bit.}$) by subtracting the SN_{exist} from the design (total) structural number (SN_{dgn-3M}).

iv) Convert the required $SN_{min-bit}$ to the required AC thickness using the appropriate structural layer coefficient of the overlay AC layer.

2) Based on Granular Base Layer Modulus:

i) Determine the structural number of surface layer for three (3) months service life (spring thawing period plus freeze-thaw cycles during November to March) (SN_{1-3M}) using the spring modulus of granular material (granular base or pulverized asphalt) layer, which exists below the bottom most bound (such as existing AC, CIR asphalt, CCPR asphalt or FDR asphalt) layer and increasing the TEF by five times.

ii) Calculate the total structural number of all bound material layer(s) (SN_{bound}) that will exist above the granular material layer, which was used to calculate the structural number of the surface layer (SN_{1-3M}). The bound materials include AC, AST, road mix, CIR asphalt, CCPR asphalt and FDR asphalt.

iii) Calculate the structural number for the minimum AC layer ($SN_{min-bit.}$) by subtracting the SN_{bound} from the SN_{1-3M}.

iv) Convert the required $SN_{min-bit}$ to the required AC thickness using the appropriate structural layer coefficient of the overlay AC layer.

3) The required minimum AC thickness is the maximum thickness from above two design analysis scenarios.

7.19 AC Thickness for Paved Shoulders

The thickness of overlay AC on existing AC paved shoulders should match with the overlay AC thickness on the main lanes. Guideline for the selection of minimum thickness of paved (AC) shoulder is presented in Table 6.0.16 (Chapter 6). The thickness of base/subbase layer(s) on a new paved shoulders should be matched with the thickness of base/subbase layer(s) on the adjacent main lane through shoulder preparation and/or bench cut, as required. GBC-I, GBC-II or GBC-M should be used to fill the thickness discrepancy between a new paved shoulder and the adjacent main lane AC, where applicable. GBC-S should be used as the surface of the unpaved portion of the shoulders including gravel shoulder rounding.

Chapter 8: DESIGN OF RIGID AND COMPOSITE PAVEMENTS FOR NEW CONSTRUCTIION AND RECONSTRUCTION

8.1 Design Inputs

The inputs for the design of rigid and composite pavements using the AASHTO 1993 Design Guide approach for new construction and reconstruction projects are:

i. Design life and ESALs
ii. Effective modulus of subgrade reaction
iii. Subgrade soils frost heave potential
iv. Pavement serviceability
v. Design reliability
vi. Overall standard deviation
vii. Overall drainage coefficient
viii. Load transfer coefficient of PCC slabs
ix. Modulus of rupture (flexural strength) of concrete
x. Modulus of elasticity of concrete

In addition to the above listed design inputs, the PCC layer's joint design including sizes of steel dowel and tie bars sizes and their placement details play critical roles in the performance of rigid and composite pavements.

8.2 Design Life and ESALs

For the new construction and reconstruction projects, rigid and composite pavements should be designed to provide a 25 years initial service life at a preselected minimum service quality without any structural enhancement, rehabilitation or major repairs such as PCC or AC overlay, extensive full depth repairs, any dowel bar retrofit to restore joint load transfer efficiency and diamond grinding of PCC surface to reduce roadway roughness. The design traffic loads i.e., the accumulative standard road repetitions or ESALs over the selected design service life should be calculated using Equation 4.1 with the appropriate TEF as presented in Chapter 4. All routes classified as trade or commerce in department's strategic classification system should be designed to handle RTAC loads regardless of traffic volume and functional classification.

8.3 Effective Modulus of Subgrade Reaction

The effective modulus of subgrade reaction (k-Value) should be calculated following the process outlined in Section 5.4 (Chapter 5).

8.4 Subgrade Soils Frost Heave Potential

For rigid and composite pavement designs, no adjustment to PCC slab thickness is required for frost susceptible subgrade soils. However, additional non-frost susceptible granular materials, meeting Manitoba's current specifications, to replace part of the frost susceptible materials or stabilization of a part of subsoil below the design subbase may be required depending on the severity (refer to Table 6.0.1, Chapter 6 for frost severity classification) of the frost heave issues. The following table (Table 8.0.1) provides a guideline for frost susceptible subsoil replacement or treatment below the subbase layer.

Table 8.0.1: Guideline for Frost Susceptible Subsoil Replacement and Treatment

Frost Severity Classification	Depth of Subsoil Replacement or Treatment Below the Subbase Layer
Severe to very severe	Remove and replace the top 300 mm of subsoil with non frost susceptible granular material or stabilize the top 300 mm of subsoil with cement (Note 1)
Negligible to Medium	None

Note 1: Increase the thickness of granular material (subbase) if the subgrade soil exhibits a resilient modulus value of less than 17.5 MPa (a soaked CBR value of less than 1.5%) to ensure long-term stable support and frost protection.

8.5 Pavement Serviceability

The recommended initial PSI (p_0) and terminal PSI (p_t) values for the design of rigid and composite pavements are presented in Table 8.0.2. The design serviceability loss due to traffic loads should be calculated as follows:

$$\Delta PSI_{TL} = p_0 - p_t \tag{8.1}$$

where,

ΔPSI_{TL} = serviceability loss due to the total traffic loads over the design service life

Table 8.0.2: Recommended Initial and Terminal Serviceability Index Values

Highway Classification	Surface Type-Texture	Initial PSI (p_0)	Terminal PSI (p_t)
Freeway, Expressway, Primary Arterial and other Trade and Commerce Routes	PCC- Tinned or Broomed Surface	4.3	2.5
	PCC- Diamond Ground Surface	4.5	2.5
	AC over PCC	4.5	2.5
Trade and Commerce Routes other than Freeway, Expressway and Primary Arterial	PCC- Tinned or Broomed Surface	4.3	2.4
	PCC- Diamond Ground Surface	4.5	2.4
	AC over PCC	4.5	2.4
Secondary Arterial and Collector (PTH and PR) and Service Road at NHS Core and Intermodal Routes	PCC- Tinned or Broomed Surface	4.3	2.3
	PCC- Diamond Ground Surface	4.5	2.3
	AC over PCC	4.5	2.3
Collector, Service (other than Service Roads at NHS Core and Intermodal Routes) and Access Roads	PCC- Tinned or Broomed Surface	4.3	2.0
	PCC- Diamond Ground Surface	4.5	2.0
	AC over PCC	4.5	2.0

8.6 Design Reliability

The recommended design reliability levels for rigid and composite pavements are provided in Table 8.0.3.

Table 8.0.3: Guidelines for the Selection of Design Reliability Levels

Highway Classification	Design Reliability, %	
	Rural x-Section	Urban and Semiurban x-Sections
Freeway	95	95
Expressway	90	90
PTHs and Trade/Commerce Routes other than freeways and expressways	85	90
Collector/Access Roads (PR/PA)	80	85

8.7 Overall Standard Deviation

For the design of rigid and composite pavements, an overall standard deviation of 0.35 should be used for all highways.

8.8 Overall Drainage Coefficient

As in the case of flexible and semi-flexible pavements, drainage and environmental conditions also affect the performance of rigid and composite pavements. In the rigid and composite pavement designs, the effect of drainage and environmental conditions are accounted for in the design using drainage coefficient, joint load transfer efficiency and appropriate PCC mixture that can withstand the environmental exposure of the PCC structures. The overall drainage coefficient depends on the quality of drainage (time required for water to drain out of the pavement structure) and the percentage of time pavement structure is exposed to moisture level approaching the saturation. In this new design guide, the effect of drainage conditions is captured using effective resilient moduli values of granular layer(s) and subgrade. In addition, the GBC- I and GBC- II materials (which are the only allowable granular base materials underneath a PCC layer) with a maximum fines content of 6% and the subbase layer materials with a low fines content are shown to provide good drainage of water from the pavement structures. As such, an overall drainage coefficient of 1.0 should be used for all designs of rigid and composite pavements.

8.9 Joint Load Transfer Coefficient

The load transfer efficiency reflects the ability of PCC pavement to distribute loads across joints and other discontinuities such as cracks in PCC slabs. In the AASHTO 1993 design method, the design joint load transfer efficiency of JPCP is accounted for in terms of joint load transfer coefficient (J-factor). It is calculated as the ratio of the deflection at the corner (at outer edge of the outer travel lane and transverse contraction joint) to the deflection at the centre of the PCC panel. The J-factor depends on whether load transfer devices are used or not, whether pavement has tied PCC shoulders, degree of aggregate interlocks at PCC joints, effective subgrade support value (k-Value), coefficient of thermal expansion of aggregates in PCC and the variation of temperature in PCC layer.

FWD deflection testing on older generation, 4.3 m wide outer panel and 225-250 mm thick, doweled PCC pavements in Manitoba has shown to provide a J-factor of 2.2 to 2.8. These PCC layers were placed over a 300 mm thick base/subbase (A base and C base materials with a high percentage of fines, low stiffness and poor drainage quality) layers underlain by very weak high plastic clay subgrade or on rubblized concrete (with a 100 mm thick granular A base layer on top of the rubblized concrete). The newer JPCP, with 4.3 m wide outer panels, 100 mm granular A base, 250 mm rubblized concrete, 125 mm granular subbase (A base and or C base) , 150 mm lime treated soil and high plastic clay subgrade, was shown to provide an average J-factor of 1.2. The average J-factor was 1.3 for a new JPCP with 32 mm dowels, 4.3 m wide outer panels and single cut (3.0 mm wide) unsealed joints placed over about 1.5 m thick GBC-I (DSB) layer and the existing PCC pavement (which was placed on a high plastic clay soil).

The AASHTO 1993 design guide recommends using a J-factor of 2.5 to 3.1 for doweled rigid (JCCP/JRCP) pavements with tied PCC shoulders and 3.2 for doweled rigid pavement with asphalt shoulders or no shoulder. Manitoba has been constructing wide (4.3 m) outer PCC panels that include 3.7 m wide travel lanes and 0.6 m monolithic PCC shoulders. These wide PCC panels are expected to provide better stress distribution than tied shoulders. However, these PCC pavements were placed on a 100 mm granular base (A-base), 200 mm subbase (C-base) and weak high plastic clay subgrade. The PCC joints were double saw cuts. A J-factor of 2.7 was used for the design of these doweled rigid and composite pavements. As stated earlier, these rigid pavements were shown to provide low values of load transfer coefficients (2.2 to 2.8). Further smaller J-factors (1.2-1.3) were recorded for PCC placed over stiffer and thicker base/subbase support.

Manitoba now specifies the use of new GBC and GSB materials, which are stiffer and more stable than the previously used A-base and C-base materials. For weak subgrade, like high plastic clay soils, the total thickness of granular material layers is also increased from 300 mm to 500 mm. Considering past practices, experienced performance in Manitoba, findings from various research sites and benefits of new stiffer and thicker GBC and GSB layers, the J-factors listed in Table 8.0.4 are recommended for the design of all doweled rigid and composite pavements (both JPCP and JRCP) including roundabouts.

Table 8.0.4: Recommended Load Transfer Coefficient for Doweled JPCP and JRCP

PCC Panel (Outer Most Travel Lane)	Total Thickness of Base and Subbase	J- Factor
≥4.3 m wide monolithic PCC panels and any lane with tied PCC shoulder	<400 mm	2.5
	400 to <500 mm	2.4
	500 to <600 mm	2.3
	600 to <750 mm	2.2
	750 to <900 mm	2.1
	≥900 mm	2.0
Standard ≤3.7 m wide panels and any sized panels in roundabouts	<400 mm	2.7
	400 to <500 mm	2.6
	500 to <600 mm	2.5
	600 to <750 mm	2.4
	750 to <900 mm	2.3
	900 to <1,050 mm	2.2
	1,050 to 1,200 mm	2.1
	≥1,200 mm	2.0

8.10 Modulus of Rupture (Flexural Strength) of Concrete

The flexural strength, a measure of tensile strength, of concrete reflects its ability to resist failure when it experiences bending or tensile stress/strain. It is determined by a third point loading test in accordance with ASTM C78/C78M: *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading*). The measured flexural strength varies from 10 to 20% of the compressive strength of concrete depending on the quality of PCC mix including the type, size, hardness, shape, texture and proportion of coarse aggregates as well as the interlock among aggregate particles. Based on test results on Manitoba PCC mixes, the average flexural strength was found to be about 13% of the compressive strength. Table 8.0.5 shows the recommended design flexural strength of different PCC mixes, when the measured flexural strength data for the project specific mix is unavailable.

Table 8.0.5: Recommended Design Flexural Strength of PCC Mixes

Design Compressive Strength	Design Flexural Strength
32 MPa (4,640 psi)	4,200 kPa (600 psi)
35 MPa (5,080 psi)	4,600 kPa (670 psi)
All other mixes	13% of the compressive strength

1 MPa = 145.038 psi

8.11 Modulus of Elasticity of Concrete

The modulus of elasticity of a PCC reflects its stiffness and ability to withstand deformation due to an applied load. It also depends on the quality of PCC mix including the quality of aggregates and mix proportions. Table 8.0.6 provides the recommended design modulus of elasticity of different PCC mixes.

Table 8.0.6: Recommended Design Modulus of Elasticity of PCC

Compressive Strength	Modulus of Elasticity
32 MPa	26,800,000 kPa (3.887 x 10^6 psi)
35 MPa	28,000,000 kPa (4.061 x 10^6 psi)
All other mixes (ACI Equation)	$E_c = 4{,}700{,}000 * \sqrt{f_c'}$

ACI = American Concrete Institute; E_c= Elastic modulus of concrete in KPa; f_c'= compressive strength of concrete in MPa; 1 MPa = 145.038 psi

8.12 Pavement Structure for New Construction and Reconstruction

The AASHTO 1993 Pavement Design Equation for rigid pavements directly provides the required thickness of the PCC layer, which is required to withstand traffic load repetitions over the design life for given subgrade support (k-Value), PCC strength properties, joint load transfer coefficient, pavement serviceability levels and design reliability. The PCC layer thickness is calculated using the following formula (AASHTO 1993):

$$log_{10}(W_{18}) = Z_R * S_o + 7.35 * log_{10}(D+1) - 0.06 + \frac{log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 * p_t) * log_{10}\left[\frac{S_c * C_d * (D^{0.75} - 1.132)}{215.63 * J * \{D^{0.75} - \frac{18.42}{(E_c/\,k)^{0.25}}\}}\right. \quad (8.2)$$

where,

W_{18} = number of standard 80 kN (18,000 lb) load repetitions (ESALs) over the Design service life

Z_R = standard normal deviate (depends on design reliability) (see Chapter 6)

S_o = overall standard deviation (0.35)

D = thickness of PCC slab (inches)

ΔPSI = serviceability loss due to traffic loads

p_t = terminal serviceability index

S_c = PCC modulus of rupture (psi)

C_d = overall drainage coefficient (1.0)

J = joint load transfer coefficient

Ec = PCC modulus of elasticity (psi)

k = effective modulus of subgrade reaction (pci)

Manitoba has been using the AASHTO DARWin software to determine the thickness of PCC layer. In the absence of DARWin software (which is no longer supported by AASHTO), Equation 8.2 can be solved for the PCC layer thickness using simple computer program and macro (e.g., MS Excel, MS Access, MATLAB). Alternatively, the design chart (Part II, Chapter 3, Figure 3.7) in the AASHTO 1993 Design Guide can be used to determine the PCC thickness.

8.12.1 Minimum Thickness of PCC Layer

The minimum net thickness of doweled PCC pavement (excluding the loss due to diamond ground texture of new PCC surface) for new construction and reconstruction of provincial highways should be 180 mm.

8.12.2 Minimum Thickness of Granular Layer(s)

The minimum thickness of the granular base/subbase layers should be selected based on the quality of subgrade and embankment materials below the base/subbase layer(s). Table 8.0.7 shows the recommended minimum thickness of base and subbase layers. Consideration should be given to use CR- M50 as subbase on highways with high traffic loads such as PTH 1, PTH

75 and PTH 190. CR- M50 must be used as subbase on highways with very high traffic loads such as PTH 100 and PTH 101 and on areas with slow/wandering traffic loads (e.g., roundabouts with more than 300 trucks per day on the design lane). The designer should consider stabilization of the top 300 mm of subgrade with portland cement as an alternative to GSB- C subbase layer, where practically and economically feasible.

On highways in flood prone areas and close to watercourses, which experience a high water pressure on the embankments, high plastic clay soil or CR-M125 rock subgrade should be used for embankment construction to restrain or minimize washout of embankments. In addition, a 300 mm thick layer of cement stabilized granular subbase layer should the placed below the granular base layer in those areas.

Table 8.0.7: Typical Minimum Thickness of Granular Base/Subbase Layer(s)

Subgrade/Embankment Soil Type	In-situ Summer Resilient Modulus	Granular Base (GBC- I or GBC-II)	Granular Subbase (CR- M50 (Note 2)
Clay, Sandy/Silty Clay, Silt or Clayey Silt	17.5 to 35 MPa (Note 1)	200 mm	300 mm (Note 3)
	>35 to 50 MPa	200 mm	250 mm (Note 3)
	>50 MPa	200 mm	200 mm (Note 3)
Silty Sand or Sand	≥60 MPa	150 mm	150 mm (Note 3)
Gravel or select granular fills	≥70 MPa	200 mm	Not required

Note 1: Increase the granular thickness if a subgrade soil exhibits a resilient modulus value of less than 17.5 MPa (a soaked CBR value of less than 1.5%) to ensure long-term stable support.

Note 2: GSB- C with a maximum of 8% fines can be used, in lieu of CR- M50, as a subbase material for highways/roads with low traffic loads (<10.0 million rigid pavement ESALs).

Note 3: Granular subbase can be replaced with granular base.

8.13 Design Examples

Example 1: Highway Information

i) Highway: A provincial four-lane divided expressway in Capital Region
ii) Highway loading classification: RTAC (NHS – Core route)
iii) Traffic volume: AADT of 25,000 with 1,700 trucks per day (1-way) and 2% annual growth rate

iv) Design service life: 25 years

v) Subgrade type and resilient modulus: High plastic clay with an effective modulus of 21.5 MPa (summer modulus of 25 MPa)

vi) Subgrade soils frost heave potential: Negligible

vii) Pavement layer materials: PCC surface, GBC- I base and CR- M50 subbase

Design Parameters

Design traffic loads (rigid pavement ESALs) with a DLF of 0.9 and TEF of 3.255 = 58,000,000
Design reliability = 90%
Initial serviceability index = 4.5 (the new PCC surface will receive a diamond ground texture)
Terminal serviceability index = 2.5
Serviceability loss due to traffic = 4.5 - 2.5 = 2.0
Overall standard deviation = 0.35
Overall drainage coefficient = 1.0
Load transfer coefficient of PCC slabs = 2.3 (for 4.3 m wide travel lane)

PCC mix properties:

28-day compressive strength = 35 MPa
28-day flexural strength = 4,600 KPa
28-day modulus of elasticity = 28,000,000 kPa

Foundation support

Granular base = 200 mm GBC- I (Equivalent Annual M_R = 224 MPa or 32,490 psi)
Granular subbase = 300 mm CR- M50 (Equivalent Annual M_R = 231 MPa or 33,510 psi)
Effective composite modulus of subgrade reaction = 87.21 kPa/mm (321 pci)

Design Layer Thickness

The required PCC thickness = 268 mm ≅270 mm
Add 10 mm to PCC design thickness to account for the loss due to diamond ground texturing of new PCC pavement surface
As such, 300 mm CR- M50, 200 mm GBC- I and 280 mm PCC layers will be required for this highway section (dowels and tie bars should be placed at 140 mm above the finished surface of granular base i.e., GBC-I layer.

8.14 Design of Composite Pavements

The design process for composite pavement is the same as the rigid pavement, except that a layer of AC will be placed as a surface layer (on the top of PCC layer). The thickness of PCC layer can be reduced from the design thickness for the added AC layer at a ratio of 1:2 i.e., 1.0 mm of PCC equals to 2.0 mm of AC. The thickness of AC layer should be 85-100 mm if the design thickness of PCC is less than 250 mm with a minimum net PCC thickness of 180 mm underlying the AC layer. If the design thickness of PCC slab is 250 mm or greater, a 100 mm thick AC layer should be placed on a 200 mm or thicker PCC layer, as required based on the design calculation. The reduced thickness of PCC layer should govern the joint design/layouts including sizes of dowels and tie bars.

Reflection cracking in the AC layer from joints and cracks in PCC layer is a challenge for composite pavement. Reflection cracking mitigation and control measures such PCC joint sealing and saw cutting in AC layer at PCC joint locations may be considered.

8.15 Design Adjustment for Organics in Subgrade or Embankment Soils

All in-situ and borrowed subgrade/embankment soils should be tested for stiffness (resilient modulus or soaked CBR) before providing the final design of pavement structures. However, the seasonal factors for the stiffness variation should be adjusted considering the moisture susceptibility of the embankment materials under consideration. For example, instead of typical 0.50, a seasonal factor of 0.40 when organic contents exceed 6% but do not exceed 10% and 0.25 when organic contents exceed 10% should be used for the spring months and very wet or saturated conditions. Consideration should be given to increase the granular subbase thickness instead of increasing the PCC layer thickness, if removal and replacement of soils containing organics is not practically feasible.

8.16 JPCP Joint Design

Joints in PCC pavements serve several functions that include accommodation of contraction and expansion of the PCC to relieve stresses that develop due to environmental changes, load transfer across adjacent slabs, isolation of structures/fixtures, lane or shoulder delineation and staging of paving operation. Joints should be placed at appropriate locations to prevent random cracking in PCC pavement slabs. PCC pavement distresses such as faulting, pumping, spalling, corner breaks, blow ups and mid-panel cracking are also developed due to improper joint design, construction and/or maintenance. As such, the use of appropriate joint types, joint layout, steel dowels and tie bars, and good construction practices including adequate consolidation of concrete mix around/at dowels, tie bars and bulkheads, concrete curing and protection, timely joint saw cutting and proper joint saw cutting or forming techniques are primary design and construction factors that contribute to satisfactory joint performance (FHWA 2019). The clear cover for any steel (dowels, tie bars, etc.) should not be less than 75 mm from the PCC top and bottom surfaces, except for some PCC overlays of existing PCC pavements. A thinner clear cover to a minimum of 60 mm can be accepted for PCC overlays if the rapid chloride permeability (RCP) of PCC is rated as very low or low (≤2,000 coulombs) when tested at 56 days.

Where a joint (contraction or expansion) with smooth dowels intersects with another joint (contraction or expansion) with smooth dowels, the placement of dowel bars in both joints should ensure a minimum of 150 mm space between transverse and longitudinal dowel bars throughout the entire length of both transverse and longitudinal dowel bars. The first dowel can be placed at up to 400 mm away from the intersection of two joints in the joint which will be subjected less load repetitions out of the two intersecting joints to meet this requirement.

There are several types of joints in PCC pavements based on their primary functions: i) contraction joints, ii) construction joints, iii) expansion joints, iv) isolation joints and v) transition joints.

8.16.1 Contraction Joints

Contraction joints in a JPCP are created to control the locations of slab cracking (i.e., to avoid random cracking) that develop due to the restraint stresses caused by moisture-related shrinkage, thermal contraction, temperature curling, and moisture warping of PCC (FHWA 2019). Contraction joints are typically created by saw cutting at a regular interval after the recently placed PCC has hardened and can be saw cut without damaging the slab and joint itself. However, such saw cut to control cracking (limit cracking at controlled locations) creates a weak vertical plane across the joint between adjacent panels. It causes a poor load distribution across the joints, especially at transverse contraction joints (joints perpendicular to the direction of traffic flow), and significantly affects PCC pavement performance. Round smooth (plain) steel dowels are used at transverse contraction joints to allow for the PCC slabs to contract freely while providing load transfer function between adjacent PCC panels. Deformed tie bars are used across the longitudinal joints (joints parallel to the direction of traffic flow) between adjacent lanes to hold the lanes together and provide load transfer across joints while limiting the cracking at controlled longitudinal locations. However, no more than three PCC lanes or 11.7 m of PCC be tied together longitudinally with deformed tie bars or deformed dowels to avoid mid-panel longitudinal cracking and joint blowups due to the expansion of PCC in JPCP.

All load transfer smooth steel dowels should be placed at 300 mm intervals, unless approved otherwise in the project details. The placement of smooth load-transfer dowels should start at 100 to 200 mm away from longitudinal joint and outer edges of PCC slabs (place 12 dowels in a 3.7 m wide panel and 14 dowels in a 4.3 m wide panel), unless approved otherwise in the project details. The diameter and length of smooth dowel bars should be selected based on the design thickness PCC layer as presented below, where the diameter refers to the diameter of steel core for all types of dowels. It should be noted that a thicker PCC is also reflective of higher truck traffic loads and the need for increased load transfer support at joints. Larger diameter dowels are also required for thicker PCC slabs because of potential high mechanical stress from traffic loads on dowels at transverse joints.

a) PCC slab thickness of 180 mm to 250 mm: 32 mm ϕ and 450 mm long
b) PCC slab thickness of 255 mm to 280 mm: 35 mm ϕ and 450 mm long
c) PCC slab thickness of 285 mm to 305 mm: 38 mm ϕ and 450 mm long

The diameter, length and spacing of deformed steel tie bars at longitudinal contraction joints will depend on roadway element and PCC thickness as listed in Table 8.0.8. A shorter spacing or larger diameter tie bars are recommended to provide added load transfer support at longitudinal joints in the following areas/scenarios:

i) between through lanes where higher number of trucks are expected to cross the longitudinal joints due to high traffic/truck volumes;

ii) for thicker PCC slabs because of potential high mechanical stress from traffic loads on deformed bars at longitudinal joints;

iii) at acceleration, deceleration, auxiliary and turning lanes including taper and cut-off, and interchange ramps/loops where higher number of trucks are expected to cross the longitudinal joints and/or slow/impact loads are experienced; and

iv) at circulatory paths of roundabouts because of potential wheel off-tracking or wandering.

Table 8.0.8: Size and Spacing of Deformed Tie Bars at Longitudinal Contraction and Construction Joints

Roadway Element	PCC Thickness	Tie Bar Diameter	Tie Bar Length	Maximum Spacing	Number of Tie Bars in a 4.5 m Long Panel
Through lanes	<280 mm	15M or Equivalent	760 mm	750 mm	5 (Note 1)
	≥280 mm	15M or Equivalent	760 mm	720 mm	6 (Note 1)
Acceleration, deceleration, auxiliary and turning lanes including taper and cut-off and interchange ramps/loops	<280 mm	15M or Equivalent	760 mm	600 mm	7 (Note 1)
	≥280 mm	20M or Equivalent	760 mm	600 mm	7 (Note 1)
Roundabouts	≤255 mm	15M or Equivalent	610 mm	500 mm	8 (Note 1)
	>255 mm	20M or Equivalent	610 mm	500 mm	8 (Note 1)

Note 1: The number of tie bars should be reduced for shorter than 4.5 m long PCC panels based on the spacing requirements. A minimum of three (3) tie bars should be placed in short slabs.

All tie bars should be spaced uniformly in each set of longitudinal joint panels. No tie bar should be placed within 450 mm of the transverse contraction joints in any case. The distance of the nearby tie bars at a longitudinal joint from any transverse joint should not exceed 750 mm. The spacing between tie bars at longitudinal contraction joints should not be less than 450 mm in any case. All dowels and tie bars should be placed at mid-depth of the net design PCC layer (excluding the thickness loss due to diamond ground texturing of a new PCC surface) taking the finished top surface of base layer as reference.

Although, theoretically significantly longer panels could be selected for thicker pavements, shorter panels (≤4.6 m in length) have shown to reduce or prevent spalling and panel cracking (FHWA 2019). Manitoba typically constructs 3.7 m wide inner (i.e., passing) lane panels and 4.3 m wide panels for outer (i.e., travel) lanes including 0.6 m monolithic PCC shoulder. As such, Manitoba adopted a maximum joint spacing of 4.5 m for any JPCP. The transverse contraction joint maximum spacing should generally be calculated as 20 times the net design thickness of PCC layer with a maximum spacing of 4.5 m (or 4.6 m in the cases of unusual joint layouts like intersections and roundabouts) for all JPCP placed on granular base.

In general, the aspect ratio between PCC panel length to width (or vice versa) should be limited to 1.25. In exceptional scenarios, an aspect ratio up to 1.5 may be accepted. Any joint layout should not create a panel with an angle of less than 60° (preferably not less than 70°) at any corner of the panel. An appropriate measure, such as slab reinforcing with steel bar mats, intermediate saw cut or drilling core holes, should be taken if the aspect ratio exceeds 1.5 and/or angle is less than 60° for any PCC panel.

The recommended depth of primary saw cut at transverse and longitudinal contraction joints vary between T/4 to T/3, where T refers to the thickness of the PCC layer. Manitoba's recommended depths of primary saw cut for various design thickness of PCC layer are specified in Table 8.0.9. The saw cut depths should be increased (from that specified in Table 8.0.9) depending on the as constructed additional PCC thickness to account for the potential loss due to diamond ground texturing of new PCC surface and/or for corrective actions for smoothness.

Table 8.0.9: Depth of Primary Saw Cuts for JPCP

Design Thickness, T (mm)	Depth of Transverse Saw Cut (mm)	Depth of Longitudinal Saw Cut (mm)
180	50	55
200	55	65
225	65	70
250	70	80
275	80	90
300	85	95
All other PCC thicknesses	(0.27 x T) to (0.29 x T)	(0.31 x T) to (0.33 x T)

T = Design thickness of PCC layer.

All transverse contraction joints should be sawn perpendicular to the centreline and at mid-length of the dowel bars. All longitudinal joints should be sawn parallel to the centreline and at mid-length of the tie bars. All transverse and longitudinal contraction joints in rural and semiurban environments should be single saw cut (3.0-3.2 mm wide) and remain unsealed, with some exceptions such as roundabouts. All transverse and longitudinal joints in urban environment and other special locations such as roundabouts should be double saw cut and sealed (depressed downward). The width of primary saw cut should be 3.0-3.2 mm. The width and depth of secondary saw cut should be selected to accommodate a Manitoba approved joint sealant product based on the sealant manufacturer's recommendation.

8.16.2 Construction Joints

A need for construction joint can be developed due to an interruption of concrete paving operation where all adjoining lanes of a roadway cannot be paved in a single pass of the concrete paver or where a new PCC is placed adjacent to an existing (old) PCC. The construction joints are then basically joints between a previously placed hardened concrete and a new PCC to be placed. Like contraction joints, construction joints can be transverse or longitudinal.

Where continual pouring of PCC is interrupted, a transverse construction joint should generally be formed at mid length of PCC slab (i.e., between two adjacent transverse contraction joints) and deformed steel dowels should be used to form a monolithic full length PCC slab. The deformed dowels should be placed at 300 mm interval c/c along the transverse construction joints. The placement of deformed dowels should start at 100 to 200 mm away from longitudinal joint and outer edges of PCC slabs. The diameter and length of deformed dowel bars at construction joints, placed at mid-panel, should be based on the design thickness of PCC layer, as specified below:

a) PCC slab thickness of 180 to 195 mm: 30M ϕ or equivalent and 610 mm long
b) PCC slab thickness of 200 mm to 280 mm: 35M ϕ or equivalent and 610 mm long
c) PCC slab thickness of ≥285 mm: 40M ϕ or equivalent and 610 mm long

If a transverse construction joint coincides with a design transverse contraction joint, in any case, smooth steel dowels as specified in previous section (Section 8.16.1) should be used. The joint saw cut and sealing requirements will apply in this case as specified in Section 8.16.1.

Deformed steel tie bars should be placed along the longitudinal joints to tie the adjacent PCC lanes together, except for the intersections and approaches at side roads. However, the total width of PCC tied together with deformed steel bars should not exceed 11.7 m (three lanes: 3.7 m + 3.7 m + 4.3 m), with some exceptions. A shorter width limit than 11.7 m will apply when the PCCP is confined with PCC curb and/or median (e.g., for roundabouts). Refer to Section 8.16.3 for expansion joints if the overall width of PCC exceeds the above specified limits.

The truck apron and curb of a roundabout should be tied with deformed steel tie bars. The depth (i.e., bottom surface) of PCC slab in apron should be matched with the depth (i.e., bottom surface) of PCC in curb at the joint of these two road features to ensure adequate cover for the tie bars. The thickening of the apron's PCC slab to meet the above specified requirement should start at a minimum distance of 600 mm from the joint of apron and curb.

The type, size and spacing of deformed steel tie bars at longitudinal construction joint will depend on roadway element and PCC thickness as specified in Section 8.16.1 and Table 8.0.8 (Section 8.16.1), except for the intersections of two roads both having PCC pavements and PCC approaches at side roads. Smooth dowels meeting the transverse contraction or expansion joint, as applies, requirements should be used at such intersections and approaches.

All dowels (deformed or smooth, as applicable) and tie bars should be placed at mid-depth of the design PCC layer taking the finished top surface of base layer as a reference.

8.16.3 Expansion Joints

American Concrete Institute (ACI) recommends that no more than three lanes (including the PCC shoulder) be tied together with deformed bars to avoid uncontrolled longitudinal cracking and blowups due to expansion of PCC (ACI 2002 and FHWA 2019). However, American Concrete Pavement Association (ACPA) recommends limiting the width of tied roadway to 14.6 m (48 ft) based on subgrade drag theory (ACPA 1992 and FHWA 2019). Based on a mechanistic analysis, Mallela et al. stated that stresses in PCC do not increase significantly when three or more lanes are tied together and experience in some U.S. states suggests that at least four lanes can be tied together without inducing uncontrolled longitudinal cracking (Mallela et al. 2009 and FHWA 2019).

Given that Manitoba experiences high day-to-night and seasonal temperature variations, it is recommended that, in general, no more than three lanes (total width of 11.7 m) including a 4.3 m wide outer panel be tied together with deformed bars. In exceptional circumstances (e.g., taper, cut-off, and acceleration, deceleration and weaving lanes), up to 14.6 m (48 ft) of PCC can be tied together with deformed bars. A shorter width limit than 11.7 m will apply when the PCCP is confined with curb, island and/or median (e.g., for roundabouts).

An expansion joint should be constructed between the inner circular path of the roadway and the inner curb of a roundabout. Another expansion joint should be constructed between the outer circular path of the roadway and the adjacent exit/entry roadway PCC slabs as well as between the outer circular path of the roadway and the outer curb of a roundabout. Expansion joint(s) may also be required at joint(s) of roadway PCC with PCC island, median curb and outer curb, depending on the total width and nature of confinement of PCC.

An expansion joint should be created to accommodate potential excessive expansion of PCC slabs when the total width of PCC exceeds the limits specified above. This would eliminate potential high compressive forces between PCC slabs or a PCC slab and any other adjacent highway structures that could result in longitudinal cracking, joint spalling and blowups or damage to the adjacent highway structures such as bridge decks and approach panels. Smooth dowels, meeting the requirements for transverse contraction joints, should be placed in the direction of potential expansion at all expansion joints to allow for the load transfer across joints and independent movement of PCC slabs. An expansion joint should be 13 mm wide; a preformed expansion joint filler should be placed covering the full depth of the PCC slab and the joint should be filled (depressed downward) with an approved joint sealant.

8.16.4 Isolation Joints

An isolation joint should be placed between PCC pavement and other fixed appurtenant structures or embedded fixtures on or adjacent to roadways such as median barrier, manhole, catch basin, utility poles and buildings to allow the PCC pavement and/or other appurtenant structures or fixtures to move independently in all directions without exhibiting damage to any of them (FHWA 2019).

Like an expansion joint, an isolation joint should be 13 mm wide, a preformed expansion joint filler should be placed to the full depth of the PCC slab and the joint should be filled (depressed downward) with an approved joint sealant. However, no load transfer dowels or tie bars are required in isolation joints. The clearance between a structure or fixture and the surrounding isolation joint should be a minimum of 300 mm. The edges of PCC pavement slabs at all sides of the isolation joints should be thickened by at least 50 mm for a design thickness of 250 mm or less and by at least 60 mm for a design thickness of greater than 250 mm with a tapered increase in thickness. The tapering to increase the slab thickness should start at a distance of 6T to 10T (T refers to the design thickness of PCC slab) from each side of the isolation joints (FHWA 2019). Consider a longer taper for thicker PCC, wherever practical.

8.16.5 Transition Joints

A transition joint is required at the intersection of two different pavement types or stages of construction to avoid differential settlement, heave or stepping. The transition joint provides gradual change in the strength of pavement structures as well as the expected distresses in pavements, and thereby provides a smoother ride experience to the travelers. The transition joint details will vary depending on the project scope such as rigid to flexible pavements or vice versa, existing to new flexible or rigid pavements, composite to rigid pavements or vice versa and composite to flexible pavements or vice versa. The Project Manager/Engineer and/or Project Design Engineer should use the appropriate standard drawings or prepare custom joint details in consultation with the Pavement Design Professional.

8.17 PCC Surface Texture

Surface texture is an important factor to provide adequate friction or skid resistance on PCC pavement surface. There are several means of texturing the PCC surface which include, but limited to, drag artificial turf or broom and then tine on the fresh PCC surface or diamond grind the hardened concrete surface. The micro-texture produced by dragging artificial turf or broom may not provide adequate and long-lasting skid resistance for a safe operation of vehicles on provincial highways (>60 km/hour speed). Therefore, in addition to providing the micro-texture, macro-texture with tinning of fresh PCC surface or diamond grinding of the hardened concrete surface is required on all highways. It should be noted that diamond ground surface provides both micro-texture and macro-texture, but micro-texturing with artificial turf or broom will be required if construction traffic is allowed on pavements before diamond ground texturing is completed. An appropriate longitudinal tinning can produce a lower noise than transverse tinning with adequate skid resistance. Transverse tinning can be considered for areas where diamond ground texturing or longitudinal tinning is not feasible due to equipment operational issues and vehicle operating speed is low (≤ 60 km/h).

In general, all new PCC pavements in Manitoba will be textured with diamond grinding of the hardened concrete surface before opening for the traffic, unless specified otherwise in the project scope or special provisions. If tinning of fresh PCC surface is considered as the macro-texturing option, potential variation of texture depth, damage to groves, non-uniform or rough PCC surface and noise issues due to improper timing and operation of tinning equipment should be taken into account before making such decision.

8.18 Thickness of AC Paved Shoulders

The thickness of base/subbase layer(s) on PCC and AC paved shoulders should match with the thickness of base/subbase layer(s) on the adjacent main lane. The thickness of PCC layer on a PCC paved shoulder should match with the thickness of PCC on the adjacent main lane. Guideline for the selection of minimum thickness of AC paved shoulder is presented in Table 6.0.16 (Chapter 6). GBC-I or GBC-II should be used to fill the thickness discrepancy between AC paved shoulder and the adjacent PCC layer. GBC-S should be used as the surface of the unpaved portion of the shoulders including gravel shoulder rounding.

Chapter 9: DESIGN FOR REHABILITATION AND PARTIAL DEPTH RECONSTRUCTION OF RIGID AND COMPOSITE PAVEMENTS

9.1 Design Process and Inputs

The rehabilitation of old generation (un-doweled or jointed reinforced) PCC pavements, placed on thin poorly drainable granular base/subbase layer(s), in Manitoba included straight AC overlay of certain minimum thickness without any structural design analysis. The rehabilitation or preservation of composite pavements included a new AC overlay with no milling or partial milling of existing AC. An unbonded PCC overlay of a composite pavement (after milling of existing AC layer and placement of a new drainable base layer) was constructed (on PTH 59 South) based on a design analysis to provide a 20 years service life. Going forward, a thorough investigation should be conducted to determine the suitability of an overlay type and material based on existing pavement condition. Recommendation should also be provided for an appropriate pre-overlay treatment for each highway/road section.

Based on the current design and construction practices of rigid pavements, Manitoba is not considering any straight AC overlay of JPCP over the life cycle of the JPCP. All PCC pavements will be maintained to the desired level of service (ride and safety) through appropriate treatments such as partial depth repairs, full depth joint repairs and full depth slab replacement to address localized distresses in PCC slabs and joints, and any localized foundation failure issues until reconstruction become the cost-effective option based on the life cycle cost analysis. Diamond grinding will be done to address faulting and roughness issues. An AC or PCC overlay can be considered after two rounds of diamond ground if the distresses in PCC slabs and joints are low such that the annualized cost of the PCC repair and overlay (AC or PCC) is less than the annualized cost of reconstruction (e.g., rubblize concrete and AC or PCC overlay) based on the life cycle cost analysis of both options. The design traffic loads (ESALs) and type (fast, slow or standing) should be taken into consideration for the selection of AC mix type, asphalt binder grade and AC layer thickness, if an AC overlay of existing PCC is selected. Unbonded PCC overlay should involve pavement investigation and structural design analysis to provide a minimum of 20 years service life.

The partial depth reconstruction of rigid and composite pavements in Manitoba typically involves rubblization of existing PCC (after milling of the existing AC surface, if any) or burying existing PCC and composite pavements with soil and/or granular materials, which are followed by a new AC or PCC surfacing. AC surfaced pavements are designed for 20 years and JPCP surfaced pavements are designed for 25 years service life.

The design process and inputs for PCC rubblization and AC overlay are the same as the AC pavement partial depth reconstruction design. The load carrying capacity i.e., structural values of the rubblized concrete and the underlying granular base/subbase as well as any new overlying granular base should be determined by assigning appropriate structural layer coefficients to these layer materials based on their conditions and properties.

The design process and inputs for an unbonded PCC overlay of an existing intact and rubblized PCC (after milling of AC, if any), buried PCC and buried composite pavements are the same as new construction of JPCP except that the determination of composite k-Value will include all support layers below the new JPCP surface.

All routes classified as trade or commerce in department's strategic classification system should be designed to handle RTAC loads regardless of traffic volume and functional classification.

9.2 Assessment of Existing Pavement

The pavement rehabilitation and partial depth reconstruction will require proper assessment of existing pavement condition and layer materials through field investigation. The department's highway inventory and pavement condition as well as maintenance databases should be thoroughly reviewed to assess pavement construction history, age, distress types and trends, and the past maintenance, preservation and rehabilitation histories including types, timing and performance. The pavement management system (PMS) outcomes for the recommended rehabilitation or reconstruction treatments, if any, should also be assessed. The suitability of any PMS recommended treatment should also be confirmed through a field investigation.

Coring should be done through the bound surface layers (AC, if any, and PCC) when conducting the soil survey on an existing pavement structure. The type and thickness of each layer material of the existing pavement structures should be determined. Samples from each type of granular material (base and subbase) should be collected and tested in the laboratory for moisture content, gradation, plasticity and classification in accordance with the department's Engineering Standard ENG- PG001 "*Soil Survey for Design and Assessment of Highway Pavements and Embankments*". Each material properties should be compared with the current and past specifications of granular base and subbase to assign appropriate structural values (structural layer coefficients or resilient moduli), as required.

All asphalt concrete and PCC cores, taken as part of site investigation, should be visually examined for the evidence of stripping, aging, degradation, scaling, ACR, ASR, etc. At least three cores from PCC layer per subsection with uniform surface condition should be tested for compressive strength. Additional cores should be taken at randomly selected crack locations and randomly selected joints to assess type and severity of cracks and AC/PCC layer(s) as well as PCC joint conditions.

The general condition of existing paved surfaces including the observed distresses should be recorded and possible reasons should be identified. Photographs of the cores, existing pavement surface, shoulders and roadsides including ditches should also be taken. The depth to the bottom of the ditches from the pavement surface should be measured. Rut depth in the AC layer of composite pavement, fault depth in existing rigid pavement and cross-fall (%) measurements should be taken on main lanes where core/boreholes are drilled. Any areas with localized unusual distresses or failures should be thoroughly investigated to determine the causes and possible measures that need to be taken.

FWD testing for determining joint load transfer efficiency, load transfer coefficient, edge support and subgrade resilient modulus should be conducted following the latest version of the Engineering Standard "*ENG- P008: Deflection Testing Using the DYNATEST® Falling Weight Deflectometer*". The deflection data should be normalized to standard load (stress) and temperature as described in the above specified standard before determining above specified parameters. Potential voids, if suspected, under the PCC slabs at joints and edges as well as the effectiveness of void sealing measures can also be determined by collecting and analyzing additional FWD deflection data in accordance with a pre-selected analysis approach.

A GPR can be used to determine the consistency in type and thickness of existing pavement layer materials and then plan for additional core/bore holes (in addition to that specified in the soil survey standard) to determine type, thickness and extent of each layer material. A GPR or MITScan can be used to locate the position as well as the alignment of dowel and tie bars and/or presence of steel mesh in PCC slabs. However, MITScan is not yet calibrated for locating non-corrosive (e.g., zinc-clad and stainless steel) dowels and tie bars that Manitoba currently uses. The process of determining the positions and alignments of dowel and tie bars using a GPR is presented in the following sections.

All the specified above information should be used to determine the feasible alternative treatments of the existing pavement.

In general, a straight AC overlay can be considered if the existing PCC slabs and joints are found to be in fair condition (compressive strength ≥24 MPa, joint LTE ≥70% and joint load transfer coefficient ≤3.2). An unbonded PCC overlay of an intact PCC pavement, with a thin inter-layer to separate them, can be considered if the existing PCC provides a compressive strength of ≥24 MPa, joint LTE of ≥50% and joint load transfer coefficient of ≤4.0. In both cases, the full depth PCC joint repair (which consists of PCC replacement at the joint for minimum length of two metres) and full depth PCC slab replacement should not exceed 15% of the total surface area. However, a life cycle cost analysis and comparison between PCC rubblization plus AC overlay and the above specified applicable alternative should be conducted to determine the most cost-effective option.

If existing PCC slabs and/or joints are in poor condition with a significant or extensive degradation and surface distress issues and/or PCC pavement exhibits joint LTE of <50% and/or joint load transfer coefficient of >4.0, PCC rubblization and AC or JPCP surfacing option should be considered.

If the existing PCC slabs and joints are in good condition with a compressive strength of ≥28 MPa and the required full depth PCC repairs do not exceed 10% of the surface area and partial depth repairs do not exceed 3% of the surface area, but joint faulting is an issue (a uniform section average faulting of >3.0 mm), dowel bar retrofit (if applicable) and diamond grinding should be considered.

A thin (40-50 mm) or ultrathin (25-35 mm) AC overlay can be considered, depending on the traffic loads, to address surface scaling and pop out issues if the PCC compressive strength is ≥32 MPa, joint LTE is ≥90% and joint load transfer coefficient is ≤2.7.

9.2.1 Determination of Dowel Bar Position and Alignment

The vertical tilt of a 450 mm long dowel can be calculated based on GPR scan data at both sides of the respective joint (e.g., 150 mm away from the centre of the joint) and using the following formula:

$$V_t = \frac{D_{-150L} - D_{+150A}}{300} x\ 450 \qquad (9.1)$$

where,

V_t = vertical tilt over the full length of the dowel (absolute value), mm

D_{-150L} = depth to dowel at 150 mm away from the centre of the joint on the leave side, mm

D_{+150A} = depth to dowel at 150 mm away from the centre of the joint on the approach side, mm

The horizontal skew of a 450 mm long dowel can be calculated using the same GPR scan data (as specified above) and the following formula:

$$Hs = \frac{H_{-150L} - H_{+150A}}{300} x\ 450 \tag{9.2}$$

where,

H_s = horizontal skew over the full length of the dowel (absolute value), mm

H_{-150L} = horizontal position of dowel at 150 mm away from the centre of the joint on the leave side, mm

H_{+150A} = horizontal position of dowel at 150 mm away from the centre of the joint on the approach side, mm

If no dowel is found at any of the above specified GPR scan locations, the dowel is missing or translated horizontally too far from the specified position.

The transverse (perpendicular to the direction of travel) translation of dowel bar from the specified position can be determined based on the mean of the dowel bar horizontal positions at H_{-150L} and H_{+150A} or with a separate GPR scan at the centre of the joint.

The vertical translation of a dowel bar from the specified depth can be determined based on the mean of the dowel bar vertical positions (depths) at H_{-150L} and H_{+150A} or with a separate GPR scan at the centre of the joint.

The longitudinal translation and absence of a 450 mm long dowel bar can be determined based on the following observation:

Table 9.0.1: Dowel Bar Longitudinal Translation Assessment

Is dowel present at following GPR scan location?				Results
-175 mm (Leave Side)	-150 mm (Leave Side)	+150 mm (Approach Side)	+310 mm (Approach side)	
Yes	Yes	Yes	No	Translation ≤50
No	Yes	Yes	No	Translation >50 to ≤75
No	No	Yes	Yes	Translation >75
Yes	Yes	No	No	Translation >75
No	No	No	No	Dowel is missing

9.2.2 Determination of Tie Bar Position and Alignment

The vertical tilt of a 760 mm long tie bar can be calculated based on GPR scan data at both sides of the respective longitudinal joint (180 mm away from the centre of the joint) and using the following formula:

$$V_{tt} = \frac{D_{180LP} - D_{180RP}}{360} x\ 760 \qquad (9.3)$$

The vertical tilt of a 610 mm long tie bar can be calculated based on GPR scan data at both sides of the respective longitudinal joint (155 mm away from the centre of the joint) and using the following formula:

$$V_{tt} = \frac{D_{155LP} - D_{155RP}}{310} x\ 610 \qquad (9.4)$$

where,

V_{tt} = vertical tilt over the full length of the tie bar (absolute value), mm

D_{180LP} = depth to tie bar at 180 mm away from the centre of the joint on the left panel, mm

D_{180RP} = depth to tie bar at 180 mm away from the centre of the joint on the right panel, mm

D_{155LP} = depth to tie bar at 155 mm away from the centre of the joint on the left panel, mm

D_{155RP} = depth to tie bar at 155 mm away from the centre of the joint on the right panel, mm

The horizontal skew of a 760 mm long tie bar can be calculated using the same GPR scan data (as specified above) and the following formula:

$$Hst = \frac{H_{180LP} - H_{180RP}}{360} x\ 760 \quad (9.5)$$

The horizontal skew of a 610 mm long tie bar can be calculated using the same GPR scan data (as specified above) and the following formula:

$$Hst = \frac{H_{155LP} - H_{155RP}}{310} x\ 610 \quad (9.6)$$

where,

H_{st} = horizontal skew over the full length of the tie bar (absolute value), mm

H_{180LP} = horizontal position of tie bar at 180 mm away from the centre of the joint on the left panel, mm

H_{180RP} = horizontal position of tie bar at 180 mm away from the centre of the joint on the right panel, mm

H_{155LP} = horizontal position of tie bar at 155 mm away from the centre of the joint on the left panel, mm

H_{155RP} = horizontal position of tie bar at 155 mm away from the centre of the joint on the right panel, mm

If no tie bar is found at any of the above specified GPR scan locations, the tie bar is missing or translated too far from the specified position.

The longitudinal (in the direction of travel) translation of a tie bar from the specified position can be determined based on the mean of the tie bar horizontal positions at H_{180LP} and H_{180RP} for 760 mm long tie bars and the mean of the tie bar horizontal positions at H_{155LP} and H_{155RP} for 610 mm long tie bars or with a separate GPR scan at the centre of the joint.

The vertical translation of a tie bar from the specified depth can be determined based on the mean of the tie bar vertical positions (depths) at H_{180LP} and H_{180RP} for 760 mm long tie bars and the mean of the tie bar vertical positions (depths) at H_{155LP} and H_{155RP} for 610 mm long tie bars or with a separate GPR scan at the centre of the joint.

The transverse translation and absence of 760 mm long tie bar can be determined based on the following observation:

Table 9.0.2: Tie Bar Transverse Translation Assessment (760 mm Tie Bars)

Is tie bar present at following GPR scan location?				Results
280 mm (Left Panel)	180 mm (Left Panel)	180 mm (Right Panel)	600 mm (Right Panel)	
Yes	Yes	Yes	No	Translation ≤100
No	Yes	Yes	No	Translation >100 to ≤200
No	No	Yes	Yes	Translation >200
Yes	Yes	No	No	Translation >200
No	No	No	No	Tie bar is missing

The transverse translation and absence of 610 mm long tie bar can be proven based on the following observation:

Table 9.0.3: Tie Bar Transverse Translation Assessment (610 mm Tie Bars)

Is tie bar present at following GPR scan location?				Results
230 mm (Left Panel)	155 mm (Left Panel)	155 mm (Right Panel)	470 mm (Right Panel)	
Yes	Yes	Yes	No	Translation ≤75
No	Yes	Yes	No	Translation >75 to ≤150
No	No	Yes	Yes	Translation >150
Yes	Yes	No	No	Translation >150
No	No	No	No	Tie bar is missing

9.3 Design for Asphalt Concrete Overlay of PCC

A straight AC overlay design of intact PCC using the AASHTO 1993 design guide approach (with rigid ESALs) yields a very thick overlay AC requirement. Reflection cracking is also a concern for any AC overlay thickness of PCC. As such, a thick AC layer based on rigid pavement design calculation is not a feasible option for Manitoba. However, a certain minimum AC overlay should be placed, after the required pre-overlay repair of existing PCC, to provide a desirable level of service with a low maintenance requirement. The existing PCC should be rubblized (after removal of existing asphalt layer, if any) if it is buried with 600 mm or less thick layer(s) of granular base/subbase/fill.

The overlay AC thickness of rubblized and buried concrete should be determined following the flexible pavement overlay design procedure (using flexible pavement ESALs) for 20 years design service life considering the PCC (intact or rubblized) as a base or subbase layer, as applicable. The AC mix type and asphalt binder grade should be selected based on 20-year design ESALs, traffic speed and site location in both cases of straight AC overlay and AC overlay of rubblized or buried PCC. The overlay AC thickness of rubblized and buried PCC can be determined following the procedure outlined below:

1) Determine the effective resilient modulus of subgrade (refer to Chapter 5).
2) Calculate the 20 years accumulative flexible pavement design ESALs (refer to Chapter 4).
3) Select the initial PSI based on the number of AC lifts to be placed.
4) Using the required all other design inputs (refer to Chapters 6 and 7), calculate the required total (design) structural number (SN_{dgn}).
5) Calculate the effective structural number (SN_{eff}) of the existing pavement layers as sum of the structural numbers of all existing pavement layers using the appropriate structural layer coefficient and thickness of each layer. Refer to Table 9.0.4 below and Table 7.0.1 (Chapter 7) for the recommended structural layer coefficients of PCC and granular base/subbase materials, respectively. Also refer to the available Engineering Standard for updates based on new research and investigation.
6) Determine the structural number of overlay (SN_{Ol}) as SN_{dgn} minus $SN_{eff.}$
7) Use the structural layer coefficients of the proposed overlay materials to determine the thickness of each overlay layer (refer to Chapter 6 for structural layer coefficients of granular and AC materials). Add 10 to 15 mm for levelling to the calculated AC thickness. Refer to Table 9.0.5 for the minimum thickness of AC layer(s). It is recommended that at least one lift (preferably two lifts) of GBC- I or

GBC- II be placed on the top of rubblized concrete before placing the AC layer to minimize potential reflection cracking issues.

Table 9.0.4: Structural Layer Coefficients of Existing PCC Materials

PCC Layer/Treatment	PCC Condition	Structural Layer Coefficient
Intact or rubblized PCC buried with native/borrowed soils	Not applicable	Ignore (consider fill material as subgrade)
Intact or rubblized PCC buried with >600 mm thick layer(s) of granular base/subbase/fill material(s)	Not applicable	Same as overlying granular material
Rubblized PCC buried with 300 to 600 mm thick layer(s) of granular base/subbase/fill material(s) (*Note 1*)	Not applicable	Same as overlying granular material
Rubblized PCC buried with <300 mm thick layer(s) of granular base/subbase material(s) (*Note 1)*	PCC with good strength (compressive strength ≥28 MPa) and/or some aging/ degradation/cracking/spalling issues	0.30
	PCC with moderate strength (compressive strength ≥24 to <28 MPa) and/or moderate aging/degradation/ cracking/spalling issues	0.25
	PCC with low strength (compressive strength ≥20 to <24 MPa) and/or significant aging/degradation/ cracking/spalling issues	0.20
	PCC with very low strength (compressive strength <20 MPa) and/or extensive aging/degradation/cracking/ spalling issues	0.15 A 200 mm (minimum) thick layer of GBC- I or GBC-II should be placed over the rubblized concrete

Note 1: The existing PCC should be rubblized (after removal of existing asphalt concrete layer, if any) if it is buried with 600 mm or less thick layer(s) of granular base/subbase/fill.

Table 9.0.5: Minimum Thickness of AC Layer

Treatment	Minimum AC Thickness
Straight AC overlay of intact PCC	<3.0 million Flexible ESALs: 80 mm 3.0 to <10.0 million Flexible ESALs: 90 mm 10.0 to <20.0 million Flexible ESALs: 100 mm 20.0 to <30.0 million Flexible ESALs: 110 mm ≥30.0 million ESALs: 120 mm
Place native/borrowed soils over intact or rubblized PCC	Based on the layered design analysis using the required minimum thicknesses of base and subbase layers (refer to Sections 6.11.2 and 6.12 in Chapter 6)
Place >600 mm thick layer(s) of granular material(s) over intact or rubblized PCC	Based on the layered design analysis (refer to Sections 6.11.2 and 6.12 in Chapter 6)
Place 300 to 600 mm thick layer(s) of granular material(s) over rubblized concrete	Based on layered design analysis (refer to Sections 6.11.2 and 6.12 in Chapter 6)
Place <300 mm thick layer(s) of granular base/subbase material(s) over rubblized concrete	140 mm (confirm the adequacy of AC thickness with layered design analysis if the thickness of granular base/subbase placed over the rubblized concrete is ≥200 mm)

<u>Asphalt Concrete Overlay Design Example: Highway Information</u>

i) Highway: A provincial four-lane divided expressway in Capital Region
ii) Highway loading classification: RTAC
iii) Traffic volume: AADT of 13,000 with 3,200 trucks per day (2-way) and 1.1% annual growth rate
iv) Design service life: 20 years
v) Subgrade type: High plastic clay
vi) Subgrade soils frost heave potential: Negligible
vii) Drainage and environmental conditions (highway context): Rural
viii) Existing pavement: 100 mm AC, 200 mm PCC (moderate strength and moderate joint degradation and reflection cracking), and 100 mm granular base
ix) New pavement layer materials: SP AC surface and GBC- I base

Design Parameters

Design traffic loads (flexible pavement ESALs) with a DLF of 0.45 and TEF of 1.421 = 16,600,000
Subgrade effective resilient modulus (based on FWD data) = 25.0 MPa (average for the section with M_R calculated for each FWD deflection test point)
Initial serviceability index = 4.4 (assume that four lifts of AC will be required)
Terminal serviceability index = 2.5
Serviceability loss due to traffic = 4.4 - 2.5 = 1.9
Overall standard deviation = 0.45
Design reliability = 90%
Structural layer coefficients of overlays: SP12.5 AC = 0.42, SP19.0 AC = 0.44, GBC- I = 0.146
Structural layer coefficients of existing layer materials: AC = N/A (mill and reclaim), rubblized concrete = 0.25 and granular base = 0.10 (resembles Granular A base)

Overlay AC Thickness
Design SN = 163.8 mm
SN of rubblized concrete = 200*0.25 = 50 mm
SN of existing base = 100*0.10 = 10 mm
SN of 100 mm (say) thick new GBC- I layer on rubblized PCC surface = 100*0.146 = 14.6 mm
SN_{OI} of required AC layer(s) = 163.8 - (50.0 + 10.0 + 14.6) = 89.6 mm
Say, 40 mm thick layer of SP12.5 AC will be used as a surface lift.
SN of 40 mm thick SP12.5 AC layer = 40*0.42 = 16.8 mm
Thickness of SP19.0 AC layer = (89.6-16.8)/0.44 = 165 mm
Add 12.5 mm for levelling; say, 180 mm thick layer SP19 AC will be placed

Recommendation: Mill and reclaim existing AC, rubblize existing PCC, and place **100 mm thick GBC- I, 180 mm thick SP19 AC and 40 mm thick SP12.5 AC layers**.

9.4 Design for PCC Overlay of PCC and Composite Pavements

A 50 mm (minimum) thick layer of asphalt treated open graded drainage layer (OGDL) should be placed as a separation layer before the placement of an unbound overlay of intact PCC (after removal of the asphalt layer in the case of composite pavement). A minimum of 100 mm thick layer of GBC- I or GBC-II should be placed over the rubblized concrete before the placement of a new JPCP layer. The existing PCC should be rubblized (after removal of the existing asphalt surface layer, if any) if it is buried with 600 mm or less thick layer(s) of granular base/subbase material(s) to ensure that no voids under the existing PCC layer is left behind.

For unbonded PCC overlay design of intact PCC or PCC surfacing over rubblized or buried PCC, the effective composite k-Value of the subgrade, existing subbase/base, existing intact or rubblized concrete base and new granular or treated base layers should be determined using the resilient/elastic moduli of all these support layers to a maximum total thickness of 900 mm. If the total thickness of existing pavement layers and new overlying granular and treated base layer(s) below a new PCC layer is 1.0 m or greater, the effective composite k-Value should be estimated using the modulus of the weakest material, considering it a subgrade placed directly below the PCC layer. The effective/equivalent annul resilient/elastic moduli of various layer materials can be selected from Table 9.0.6. Refer to Chapter 5 for the determination of effective composite k-Value. Using this effective composite k-Value and all other inputs as specified for a new JPCP in Chapter 8, the required thickness of the PCC layer can be determined following the procedure described in Chapter 8. However, a lower J factor (recommended J factor = 2.0) should be used for the unbonded PCC overlay of intact PCC. The thickness of the PCC layer should in no case be less than 150 mm for unbound PCC overlay of intact PCC and 180 mm when the existing PCC is rubblized and/or buried.

<u>Unbonded PCC Overlay Design Example: Highway Information</u>

i) Highway: A provincial four-lane divided expressway in Capital Region
ii) Highway loading classification: RTAC
iii) Traffic volume: AADT of 15,000 with 1,025 trucks per day (2-way) and 1.7% annual growth rate
iv) Design service life: 20 years (unbonded PCC overlay)
v) Subgrade type and resilient modulus: High plastic clay with an effective resilient modulus value of 27.2 MPa (3,945 psi)
vi) Subgrade soils frost heave potential: Negligible
vii) Existing pavement: 175 mm AC, 200 mm PCC (moderate strength and significant joint degradation and reflection cracking), and 100 mm granular base
viii) Pavement layer materials: Unbonded PCC overlay with an asphalt treated OGDL inter-layer.

Table 9.0.6: Elastic/Resilient Modulus of Support Layers

Material	Resilient/Elastic Moduli
Existing subgrade	Refer to Chapter 7
Existing granular	Refer to Chapter 7
OGDL	350,000 psi
Intact PCC for unbound PCC overlay using OGDL as inter-layer	1,000,000 psi (consider the intact PCC as a lean concrete subbase)
New GBC- I or GBC- II	Refer to Chapter 6
Intact or rubblized PCC buried with native/borrowed soils	Ignore the contribution of intact or rubblized PCC
Intact or rubblized PCC buried with >600 mm thick layer(s) of granular base/subbase/fill material(s)	Same as overlying granular layer material
Rubblized concrete buried with 300 to 600 thick layer(s) of granular base/subbase/fill material(s) (consider the rubblized concrete a granular subbase layer)	Same as overlying granular layer material.
Rubblized concrete buried with <300 mm thick layer(s) of granular base/subbase material(s) (consider rubblized concrete as a treated or granular subbase layer depending on its elastic modulus value)	PCC with good strength (compressive strength ≥28 MPa) and some aging/degradation/cracking/spalling issues: 900 MPa (130,000 psi) PCC with moderate strength (compressive strength ≥24 to <28 MPa) and moderate aging/ degradation/cracking/spalling issues: 585 MPa (85,000 psi) PCC with low strength (compressive strength ≥20 to <24 MPa) and significant aging/degradation/ cracking/spalling issues: 370 MPa (53,600 psi) PCC with very low strength (compressive strength <20 MPa) and extensive aging/degradation/ cracking/spalling issues: Granular material with a M_R of 230 MPa (33,600 psi)

Design Parameters

Design traffic loads (rigid pavement ESALs) with a DLF of 0.45 and TEF of 2.263 = 9,000,000
Design reliability = 90%
Initial serviceability index = 4.5 (diamond ground textured surface)
Terminal serviceability index = 2.5
Serviceability loss due to traffic = 4.5 - 2.5 = 2.0
Overall standard deviation = 0.35
Overall drainage coefficient = 1.0
Load transfer coefficient of PCC slabs = 2.0

<u>PCC mix properties:</u>
28-day compressive strength = 32 MPa
28-day flexural strength = 4,200 KPa
28-day modulus of elasticity = 26,800,000 KPa

<u>Treatment of Existing Pavement:</u>
Mill and reclaim AC, mill PCC at degraded joints and fill the milled joints with asphalt concrete.

<u>Foundation Support</u>
OGDL = 50 mm (equivalent annual M_R = 350,000 psi)
Existing PCC = 200 mm (equivalent annual M_R = 1,000,000 psi)
Existing granular base = 100 mm limestone (equivalent annual M_R = 140 MPa or 20,300 psi)
Effective composite modulus of subgrade reaction = 509 pci (138.17 kPa/mm)

Design PCC Layer Thickness

The required PCC thickness = 177 mm; say, 180 mm.
With a 35 MPa PCC, the required thickness is 166 mm; say, 170 mm.
Add 10 mm extra for the diamond ground texturing of the new PCC pavement.
The recommended unbonded PCC overlay thickness = 190 mm for a 32 MPa PCC mix or 180 mm for a 35 MPa PCC mix.

9.5 JPCP Joint Design

Refer to Chapter 8 for the details of PCC joints and dowels as well as tie bars with the exception for unbonded PCC overlays of thickness ≤190 mm with rigid pavement design ESALs of less than 10 million. For the unbonded PCC overlay of thickness ≤190 mm with rigid pavement design ESALs <10 million, no load transfer dowels are required at the transverse contraction joints. 25M ϕ or equivalent and 610 mm long deformed dowels can be used at transverse construction joints when the unbonded PCC overlay thickness is 150 to 175 mm. A shorter joint spacing (say, 20 times the design thickness of PCC layer) should be considered for unbonded PCC overlays than the standard spacing, which is used for new construction or reconstruction (PCC placed on granular base) projects.

Appropriate tie bars should be used at the longitudinal joints (refer to Chapter 8). The clear cover for any steel (smooth/deformed dowels, tie bars, mesh) should not be less than 75 mm from any PCC surface (top or bottom) in any case, unless the rapid chloride permeability of PCC is rated as very low or low (≤2,000 coulombs at 56 days). A thinner clear cover to a minimum of 60 mm can be accepted for unbonded PCC overlays if the chloride permeability of PCC is rated as very low or low.

9.5 PCC Surface Texture

Refer to Chapter 8

9.6 Thickness of AC Paved Shoulders

The thickness of PCC layer on a new PCC paved shoulder should match with the thickness of PCC layer on the adjacent main lane. The thickness of overlay AC on an existing AC or PCC paved shoulder should match with the overlay AC thickness on the main lanes. Guideline for the selection of minimum thickness of new AC paved shoulder is presented in Table 6.0.16 (Chapter 6). The thickness of base/subbase layer(s) on new PCC and/or AC paved shoulder should be matched with the thickness of base/subbase layer(s) on the adjacent main lane through shoulder preparation and/or bench cut, as required. GBC-I or GBC-II should be used to fill the thickness discrepancy between new paved shoulder and the adjacent main lane AC or PCC thickness, as applicable. GBC-S should be used as the surface of the unpaved portion of the shoulders including gravel shoulder rounding.

Chapter 10: DESIGN OF GRAVEL SURFACED PAVEMENT FOR NEW CONSTRUCTION AND RECONSTRUCTION

10.1 Introduction

Manitoba's gravel (aggregate) surfaced pavements generally consist of 25 mm to 150 mm thick layer of granular aggregates placed on untreated subgrade soils, except for some heavy haul (e.g., resource) roads where thick subbase and/or base layer(s) are being placed before placing the surface aggregates (gravel). Typically, the thickness of such granular aggregate layers has been selected by regional staff considering availability of materials, costs, site conditions and traffic loads (in some cases) without any form of design analysis. Maintenance treatments such as addition of new gravel and regrading are being done to keep these roads in safe driving condition. Pavement designs for gravel roads have mainly been provided for some special circumstances such as high traffic service roads and local/access roads that may be paved with AST or AC in near future.

Gravel surfaced roads with thick subbase/base layers are shown to provide better serviceability with reduced maintenance activities. As such, it is recommended that a certain minimum granular base/subbase should be placed below the surface aggregate (traffic gravel) to reduce the maintenance costs and improve safety as well as sustainability. The roads that will probably be paved within the next five years should be properly designed to accommodate the surfacing AC layer without doing full depth reconstruction and/or grade widening. This will significantly reduce the construction costs in Phase II (i.e., AC surfacing).

In the AASHTO 1993 Design Guide, the design of low volume gravel (aggregate) surfaced roads is based on the acceptable serviceability loss and the acceptable rutting. Nomographs have been provided to determine the allowable axle load repetitions in each climatic season for a range of estimated granular base thickness based on the above stated both criteria. The subgrade resilient modulus in each of those climatic seasons and the elastic modulus of base material are other input parameters. The procedure involves calculation of damage in each climatic season and then the total damage for each estimate of granular base layer thickness. The calculation process must be repeated for four estimates of base thickness to develop a curve of total damage against the estimated base layer thickness for both of the serviceability and rutting criteria. The maximum thickness from these two curves, corresponding to the damage factor of 1.0, will be taken as the initial design granular base thickness. The granular base thickness then should be adjusted for potential loss of gravel. A nomograph also is provided to convert a part of the granular base thickness to subbase layer thickness. The whole process is cumbersome and some parameters for Manitoba's typical subgrade falls outside the range in the nomographs. The gravel loss can also vary widely from area to area or road section to road section. This procedure has never been used and validated in Manitoba. As such, Manitoba has adopted the same design procedure and required input parameters as the new construction or reconstruction design of flexible or semi-flexible pavements for gravel road new construction and reconstruction designs using the AASHTO 1993 Design Guide approach.

10.2 Design Life and ESALs

For a new construction or reconstruction project, gravel road pavements should be designed to provide a 20 years initial service life at a preselected minimum service quality. The design traffic loads i.e., the accumulative standard load repetitions or ESAls over the selected design service life should be calculated using Equation 4.1 with the appropriate TEF as outlined in Chapter 4. The design ESALs should be a minimum of 10,000 for gravel surfaced roads as recommended in AASHTO 1993 design guide. Refer to Chapter 6 for the minimum design ESALs for roads that will be paved within next five years.

10.3 Subgrade Soil Stiffness

Refer to Chapter 5.

10.4 Subgrade Soils Frost Heave Potential

Refer to Chapter 6 if frost heave is a consideration in the design based on the available budget for construction or reconstruction.

10.5 Design Serviceability Loss Due to Frost Heave

Refer to Chapter 6, if frost heave is a consideration in the design.

10.6 Pavement Serviceability

Tables 10.0.1 and 10.0.2 present the guidelines for the selection of p_0 and p_t values, respectively.

Table 10.0.1: Guideline for Initial Serviceability Index (p_0)

Strategy	Initial PSI (p_0)
Surface will be paved with AST within next five years or will remain gravel	4.0
Surface will be paved with AC within next five years	4.2

Table 10.0.2: Guideline for Terminal Serviceability Index (p_t)

Strategy	Terminal PSI (p_t)
Surface will be paved with AC or AST within next five years	Same as flexible or semi-flexible pavement, as applicable (refer to Chapter 6)
Surface will remain gravel	1.0

10.7 Design Reliability

Table 10.0.3 presents the guidelines for the selection of design reliability levels.

Table 10.0.3: Guidelines for the Selection of Design Reliability, %

Highway Classification	Surfacing Strategy	Design Reliability, %	
		Rural x-Section	Urban and Semiurban x-Sections
Collector/Access Roads (PR/PA)	Surface will be paved with AC or AST within next five years	Same as flexible or semi-flexible pavement, as applicable (refer to Chapter 6)	
Collector/Access Roads (PR/PA)	Surface will remain gravel for unforeseeable future	50	60

10.8 Overall Standard Deviation

Use an overall standard deviation of 0.45.

10.9 Drainage and Environmental Conditions

Refer to Chapter 6.

10.10 Pavement Layer Materials Properties

Refer to Chapter 6.

10.11 Pavement Structure for New Construction and Full Depth Reconstruction

Refer to Chapter 6 (Section 6.11) for the determination of the design (total) structural number (SN_{dgn}). The total i.e., SN_{dgn} then should be converted into layer thickness of different materials to be used in the actual construction.

10.11.1 Selection of Layer Thicknesses

The total structural number (SN_{dgn}) can be converted into thicknesses of different layer materials using the effective layer coefficient (a) values of the materials that are to be used in the actual construction of pavements. Equation 10.1 can be used for the determination of layer thickness of each material. It should be noted again that drainage coefficient (m) is not required when using the effective structural layer coefficients.

$$SN_{dgn} = \sum D_i a_i \tag{10.1}$$

where,

D_i = thickness of layer i (1, 2, 3, 4……….)

a_i = structural layer coefficient value of layer i (1, 2, 3, 4……….)

For the design of a gravel road that will be paved with AC within the next five years, calculate the thickness of granular base or thicknesses of granular base and subbase considering that a 100 mm thick layer of AC will be constructed within next five years. For the interim stage (Phase I) construction (i.e., interim surfacing), consider the placement of a 100 mm thick GBC-S surface layer, in lieu of 100 mm thick AC layer, over the granular base layer. Confirm that the gravel pavement structure is adequate to carry traffic loads for the next years using the design reliability and serviceability indices as applicable to gravel surfaced roads. Increase the granular base and/or subbase thickness, if required, to meet the design requirements for five years service life.

When the region is ready to pave the road, they should scope the work as remove 100 mm GBC-S, regrade and re-compact the GBC surface and place 100 mm AC, unless an updated estimate of traffic loads requires a thicker AC layer. In the interim stage construction, the initial grade should be wide enough to accommodate an additional 100 mm thick lift of GBC in Phase II construction, if there is a potential for significant increase in truck traffic volume within the next 20 years.

10.11.2 Design Examples

<u>Example 1 (Low Volume Gravel Road): Highway Information</u>

a) Highway: A provincial undivided 2-lane collector highway in Capital Region (climate zone 1)
b) Highway loading classification: B1
c) Traffic volume: AADT of 100 with 20 trucks per day (2-way) and 0.5% annual growth rate
d) Design service life: 20 years
e) Subgrade type and resilient modulus: High plastic clay with a summer resilient modulus of 25 MPa
f) Subgrade soils frost heave potential: Negligible
g) Drainage and environmental conditions (highway context): Rural
h) Pavement layer materials: GBC- S surface, GBC- M base and GSB- C subbase

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.5 and TEF of 0.80 = 61,200
Subgrade effective resilient modulus = 21.5 MPa
Design reliability = 50%
Initial serviceability index = 4.0 (rural, will remain unpaved)
Terminal serviceability index = 1.0 (will remain unpaved)
Serviceability loss due to traffic (no loss due to frost) = 4.0 – 1.0 = 3.0
Overall standard deviation = 0.45
Structural layer coefficients: GBC- S = 0.104, GBC- M = 0.129 and GSB- C = 0.123

Design SN and Layer Thickness

The calculated total (design) SN_{dgn} = 60.0 mm
Assume that a 100 mm thick layer of GBC- S will be used

SN of surface layer (GBC-S) = 100*0.104 = 10.4 mm
Select base (GBC- M) layer thickness; say, a 100 mm thick layer of GBC- M will be used
SN of base layer (SN_2) = 100 * 0.129 = 12.9 mm
SN of subbase layer (SN_3) = SN_{dgn} - (SN_1+ SN_2) = 60.0 - (10.4 + 12.9) = 36.7 mm
The required thickness of subbase layer (GSB- C) = 36.7/0.123 = 298 mm; say, a 300 mm thick layer of GSB-C will be used.

Then the required pavement structure is: **100 mm thick GBC- S (surface), 100 mm thick GBC- M and 300 mm thick GSB- C.**

Example 2 (Low Volume Gravel Road): Highway Information

a) Highway: A provincial undivided 2-lane collector highway in Capital Region (climate zone 1)
b) Highway loading classification: B1
c) Traffic volume: AADT of 200 with 50 trucks per day (2-way) and 1.0% annual growth rate
d) Design service life: 20 years
e) Subgrade type and resilient modulus: High plastic clay with a summer resilient modulus of 25 MPa
f) Subgrade soils frost heave potential: Negligible
g) Drainage and environmental conditions (highway context): Rural
h) Pavement layer materials: GBC- S surface, GBC- M base and GSB- C subbase. Surface will be replaced with AC (Bit. B) within the next five years.

Design Parameters

Design traffic loads (ESALs) with a DLF of 0.5 and TEF of 1.02 = 205,000
Subgrade effective resilient modulus = 21.5 MPa
Design reliability = 80%
Initial serviceability index = 4.2 (two lifts of Bit. B will be placed within the next five years)
Terminal serviceability index = 2.1
Serviceability loss due to traffic (no loss due to frost) = 4.2 – 2.1 = 2.1
Overall standard deviation = 0.45
Structural layer coefficients: Bit. B = 0.40, GBC- S = 0.104, GBC- M = 0.129 and GSB- C = 0.123

Design SN and Layer Thickness

The calculated total (design) SN_{dgn} = 84.9 mm
Assume that a 100 mm thick layer of Bit. B will be placed within the next five years.
Effective SN of the surface layer (100 mm thick Bit. B, excluding 12.5 mm for levelling) = (100-12.5)*0.40 = 35.0 mm
Select base (GBC- M) layer thickness; say, a 100 mm thick GBC- M layer will be used
SN of base layer (SN_2) = 100 * 0.129 = 12.9 mm
SN of subbase layer (SN_3) = SN_{dgn} – (SN_1 + SN_2) = 84.9 – (35.0 + 12.9) = 37.0 mm
The required thickness of subbase (GSB- C) layer = 37.0/0.123 = 301 mm; say, a 300 mm thick layer of GSB-C will be used.

Check that the gravel pavement structure is adequate to carry traffic loads for the next five years using initial serviceability index of 4.0, terminal serviceability index of 1.0, design reliability of 50% and design traffic loads over five years (47,500 ESALs)
Calculated SN_{dgn} for gravel road service life of five years = 57.8 mm
SN of initial gravel pavement structure = 100 * 0.104 + 100 * 0.129 + 300 * 0.123 = 60.2 mm > SN_{dgn}, design is good.

Interim stage (Phase I) construction: **100 mm thick GBC-S**, **100 mm thick GBC-M** and **300 mm thick GSB-C layers.**

The initial grade should be wide enough to accommodate additional 100 mm GBC-M in Phase II if truck volume is expected to increase significantly within the next 20 years.

Phase II construction: Remove GBC-S layer, place 100 mm GBC-M (if warranted) and place a 100 mm thick Bit. B layer

10.12 Pavement Analysis and Design for Frost Heave Management

Refer to Chapter 6, if required.

10.13 Design Adjustment for Organics in Subgrade or Embankment Soils

Refer to Chapter 6, if required.

10.14 Thickness of AC Paved Shoulders

Refer to Chapter 6, if required.

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