ISBN: 978-1-4435-2872-6 (Print)
ISBN: 978-1-4435-2873-3 (PDF)

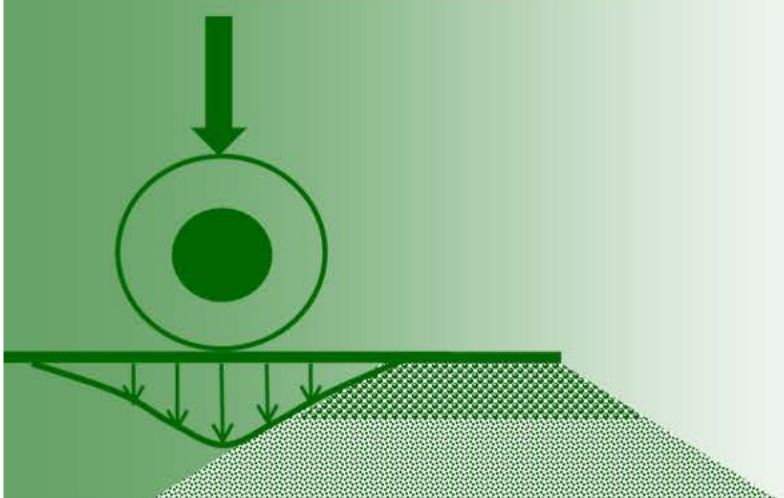
Ontario

Ministry of Transportation

Pavement Design and Rehabilitation Manual

Material | Drainage | Design | Preservation

Rehabilitation | Maintenance | Evaluation | Management



Second Edition

2013

MTO MERO

Materials Engineering and Research Office

(Blank Page)

PAVEMENT DESIGN AND REHABILITATION MANUAL

Second Edition

Published by:
Materials Engineering and Research Office
Ministry of Transportation
1201 Wilson Avenue
Downsview, Ontario
Canada M3M 1J8

ISBN: 978-1-4435-2872-6 (PRINT)
978-1-4435-2873-3 (PDF)

March 2013

(Blank Page)

To all users of the: **PAVEMENT DESIGN AND REHABILITATION MANUAL**

Enquiries regarding the purchase and distribution of this Manual should be directed to:

Publications Ontario
880 Bay Street
Toronto, Ontario, Canada M7A 1N8

http://www.publications.serviceontario.ca/ecom/

Enquiries regarding amendments, suggestions or comments should be directed to:

Materials Engineering and Research Office
Ministry of Transportation Ontario
2nd Floor, Building C
1201 Wilson Avenue
Downsview, ON
M3M 1J8

(Blank Page)

Foreword

This Second Edition of the MTO Pavement Design and Rehabilitation Manual is a publication of the Ontario Ministry of Transportation for use in the Province of Ontario.

The Manual incorporates the ministry's current practices and expertise in the areas of pavement design, rehabilitation, and management.

This Manual provides a documented set of guidelines and procedures for use by ministry planners, designers and policy makers. Use of these design practices will assist in providing cost-effective pavement design and uniform direction in the preservation of our pavement infrastructure. This second edition incorporated the newly adopted information on SuperPave, the AASHTOWare Pavement ME Design, warm mix asphalt, pavement preservation and other new pavement design and rehabilitation technologies.

Pavement design is steadily changing from an art to a science but is still dependent on the experience and good judgement of the designer. The Manual is not a substitute for experience and understanding in this specialized field of geotechnical engineering.

The information presented in this Manual was carefully researched and presented. However, no warranty, express or implied, is made on the accuracy of the contents or their extraction from referenced publications; nor shall the fact of distribution constitute responsibility by the Ontario Ministry of Transportation, or any researchers or contributors for omissions, errors or possible misrepresentation, or financial loss that may result from use or interpretation of the material herein contained. Specific construction and material related issues should be referred to Ontario Standard Special Provision (OPSS) for details, which can be found from the following website:
http://www.raqsa.mto.gov.on.ca/techpubs/OPS.nsf/OPSHomepage

Comments and suggestions on the technical contents of the Manual are welcome. Such comments should be addressed to:

Head
Pavements & Foundations Section
Materials Engineering and Research Office
Ministry of Transportation
1201 Wilson Avenue
2nd Floor, Building C
Downsview, Ontario
Canada M3M 1J8

(Blank Page)

Manual Steering Committee

Team Lead – **Tom Kazmierowski**, Former Manager of Materials Engineering and Research Office

Members – **Becca Lane**, Manager of Materials Engineering and Research Office
Betty Bennett, Senior Pavement Design Engineer
Chris Raymond, Head of Construction Contracts
Dale Smith, Team Lead of Performance
Dan Schutte, Head of Program Planning (Northwestern Region)
Darwyn Sproule, Head of Planning and Design (Eastern Region)
Nick Gilbert, Head of Geotechnical Section (West Region)
Stephen Lee, Head of Pavements and Foundations
Susanne Chan, Pavement Design Engineer

(Blank Page)

Technical Working Committees

Chapter 1: Materials

Chairman: Susanne Chan, Pavement Design Engineer

Members:
Anil Virani, Senior Bituminious Engineer
Betty Bennett, Senior Pavement Design Engineer
Chris Raymond, Head of Construction Contracts
Maria Bianchin, Lead of Performance Specifications Task Group
Robert Mount, Geotechnical Engineer (West Region)
Steve Senior, Head of Soils and Aggregates

Chapter 2: Drainage

Chairman – Susanne Chan, Pavement Design Engineer

Members:
Betty Bennett, Senior Pavement Design Engineer
Chris Raymond, Head of Construction Contracts
Hani Farghaly, Senior Engineer of Hydrotechnical Design
Rob Kohlberger, Geotechnical Engineer (Central Region)
Steve Senior, Head of Soils and Aggregates

Chapter 3: Pavement Design

Chairman – Susanne Chan, Pavement Design Engineer

Members:
Betty Bennett, Senior Pavement Design Engineer
Chris Raymond, Head of Construction Contracts
Dale Smith, Team Lead of Performance
Derek Thompson, Geotechnical Engineer (Northeastern Region)
Jason Wright, Geotechnical Engineer (Northeastern Region)
Joseph Ponniah, Former Senior Research Engineer
Joshua Li, Pavement Design Evaluation Officer (Central Region)
Maria Bianchin, Lead of Performance Specifications Task Group
Mark Ayton, Senior Engineer Highway Design
Muhammad Kamran Khan, Project Soils Engineer (West Region)
Pamela Marks, Head of Bituminous Section
Rob Tardif, Team Leader Transportation Planning
Steve Senior, Head of Soils and Aggregates
Warren Lee, Pavement Design Engineer

Chapter 4: Maintenance, Preservation and Rehabilitation

Chairman – Susanne Chan, Pavement Design Engineer

Members:
Betty Bennett, Senior Pavement Design Engineer
Brij Sharma, Pavement Management Engineer
Chris Raymond, Head of Construction Contracts
Marc Rockburn, Pavement Design Evaluation Officer (Northeastern Region)
Maria Bianchin, Lead of Performance Specifications Task Group
Pamela Marks, Head of Bituminous Section
Steve Senior, Head of Soils and Aggregates
Ted Phillips, Former Geotechnical Supervisor (Eastern Region)

Chapter 5: Evaluation and Pavement Management

Chairman – Susanne Chan, Pavement Design Engineer

Members:
Betty Bennett, Senior Pavement Design Engineer
Brij Sharma, Pavement Management Engineer
Chris Raymond, Head of Construction Contracts
Frank Marciello, Supervisor of Pavement Evaluation Unit
John Blair, Senior Soils and Aggregates Engineer
Li Ningyuan, Senior Pavement Management Engineer
Nick Gilbert, Head of Geotechnical Section (West Region)
Warren Lee, Pavement Design Engineer

Acknowledgements

The MTO Pavement Design and Rehabilitation Manual manifest the talents and dedication of the ministry staff from various offices. The actual publication of this document is a complex and time-consuming task made easier by the enthusiasm of the people involved. Their contributions are gratefully acknowledged.

Project Manager – Susanne Chan, Pavement Design Engineer
Technical Editing – Technical Working Committee Members
Independent Reviewer – Alison Bradbury, Former Head of Engineering Standards and Specification Management Section
Proofreading – Steering Committee Members

Table of Contents

Chapter 1:

MATERIALS

1 Materials

1.1 SUBGRADE MATERIALS

1.1.1 SOIL AND ROCK IDENTIFICATION AND CLASSIFICATION

Physiography of Ontario

The physiography and topography of Ontario are shaped by bedrock formations of Precambrian and Paleozoic ages, and the soil deposits placed during and after the ice ages of the last 130,000 years. The bedrock formations, apart from a few exceptions, are naturally divided into two geographic regions by a boundary approximately joining Penetanguishene and Kingston. North of the boundary, the topography is dominated by ridges and hills of granite and metamorphic rock of Precambrian age. South of the line, the Precambrian rock is overlain by flat, softer limestone, shale and sandstone of Paleozoic age. The Precambrian rock in the Hudson Bay Lowlands, and in the area northeast of Kingston is also overlain by Paleozoic rock.

Soil deposits of variable thicknesses overlie bedrock throughout most of Ontario and are generally of glacial origin. Glacial till was deposited at the base of retreating thick ice sheets and consists largely of unsorted and heterogeneous mixtures of sand, silt, and clay, cobbles, and boulders. During intermediate and final retreats of the glaciers, large quantities of melt water were released which resulted in the deposition of sorted sands and gravel near the ice sheets, in outwash plains, valley trains and eskers. Silts and clays were deposited further away in the large lakes dominating the early post glacial period. Therefore, soil deposits in Ontario often consist of lacustrine (lake deposits) silts and clays, and fluvial (river deposits) sands and gravels overlying a till sheet which, in turn, overlies bedrock. Interbedding of the till with the other deposits is also often found and is the result of intermediate retreats of the glaciers.

Further information about the geology and stratigraphy of Ontario may be found in the following website:

www.geologyontario.mndm.gov.on.ca

Published sources which may also be referred to are:

- The Physiography of Southern Ontario [1]
- The Ontario Soil Survey (Canada Department of Agriculture) [2]
- Numerous publications of the Ontario Geologic Survey [3]
- Water well records [4]

http://www.ene.gov.on.ca/en/publications/dataproducts/index.php#PartLand

The Pavements and Foundations Section maintains a GeoCRES library of borehole logs and subsurface information from ministry field investigations conducted since the 1950's.

Soil Classification

Soil classification provides a standard method of identifying and grouping soils based on their physical characteristics and engineering properties. Soils in a particular group have similar characteristics and exhibit similar behaviour that enables a designer to develop appropriate engineering solutions.

Soil properties are determined by visual examination and laboratory testing. Visual assessment includes particle size and shape, colour, moisture content and relative strength. Visual examination is supplemented by laboratory testing when more accurate information is required. Laboratory testing usually includes gradation, moisture content, Atterberg Limits to measure plasticity, and Proctor moisture/density.

Soils are classified into one of eighteen soils groups based on their visual characteristics and engineering properties. These procedures are described in detail in Appendix A.

The Soil Classification Manual, included herein as Appendix A, describes ministry procedures for the description and classification of soils.

Soil Descriptions

Soils are often described using abbreviations. Standard abbreviations to expedite fieldwork are provided in Appendix A.

Example: A grey silty clay with sand, trace of fine gravel, moist, medium plasticity would be abbreviated to "Gry Si(y) Cl W Sa, Tr F Gr, moist, MP"

The principal and secondary components are initially estimated and described, and then the minor components are identified. These minor components are described as follows:

with describes a component having a significant effect on the sample

some describes a component having a moderate effect on the sample

trace describes a component having only a minor effect on the sample

Further refinements in identifications can be made on the basis of laboratory tests. The descriptive terms, based on percent by mass of the whole sample, are described as follows:

Descriptive Term	**Example**	**Percent by Mass of Sample**
and (with two major soil types)	sand and gravel	40-60
adjective (silty)	silty	30-40
with	silt, with fine sand	20-30
some	silt, some fine sand	10-20
trace	sand, trace of gravel	0-10

The Ministry Soil Classification System describes four types of organic soils. Three of the fine grained soil groups are described as organic (O Group). These groups are silt or clay-sized soils having a relatively low plasticity which plots below the A line on the soil plasticity chart. The O Group is differentiated from the Highly Organic Peats (Pt Group). The Pt Group is identified by its dark colour, earthy odour, spongy consistency and frequently by its fibrous texture. Peat may be subdivided into three general classes, as shown below. The engineering properties of these three types of peat are significantly different and therefore require an accurate identification.

Amorphous	Formless, highly decomposed, commonly called muck. No resistance to tearing.
Fine Fibrous	Predominantly small or thin or non-woody fibres which offer some resistance to hand tearing.
Coarse Fibrous	Predominantly small or thin or woody fibres which offer relatively high resistance to hand tearing.

Engineering Properties

Each of the eighteen soil groups has a unique set of engineering properties and performance characteristics. The performance characteristics give a general idea of the soil behaviour during and after construction. The engineering properties include:

a) Permeability when compacted
b) Strength when compacted
c) Compressibility when compacted
d) Workability as a construction material
e) Scour resistance
f) Susceptibility to surficial erosion
g) Susceptibility to frost action
h) Drainage characteristics

The engineering properties of the soil groups is presented in tabular form in Appendix A.

Rock

Pavement investigations seldom require core drilling and sampling or geophysical techniques to prove bedrock. The top of bedrock is most commonly identified as the depth of refusal to auger advancement. Augers can, however, encounter refusal on boulders, and this can be misinterpreted as top of bedrock. It is recommended that additional test holes be advanced in areas where refusal to augers occurs at variable depths and elevations in order to confirm bedrock. Refer to MERO Report # 30 [5] for details.

Shale is a fine-grained, low strength, low durability sedimentary rock. It is typically horizontally bedded and consists of approximately 15% sand, 50% silt, and 35% clay-size particles. Shale can range in consistency from very soft soil-like material to hard, unweathered rock. Some shale formations (e.g., Queenston Formation) are composed almost entirely of shale. Others (e.g., Georgian Bay Formation) contain interbedded hard limestone layers. An Ontario Shale Rating System was developed based on slake durability, plasticity index, and point load strength. The document “Evaluation of Shales for Construction Project” [6] provides detailed descriptions of the major shale formations of Ontario, their engineering properties, and the various engineering uses of shale. The document “Shale Fill Construction Practices” [7] has additional information on shale.

On exposure to cycles of wetting and drying, shale undergoes rapid deterioration by a process called slaking. As a result of this slaking process, shale represents a unique engineering material that cannot be readily categorized as earth or rock. OPSS 206 defines shale as rock, and shale excavation is included in the Rock Excavation item. Because of its peculiar properties, it is essential to differentiate between shale and other rock in the Pavement Design Report and on the soils profile.

In Ontario, most 'other rocks' that make up greater than 90% of bedrock in Ontario (including granite, gneiss, quartzite, limestone, and sandstone) range in strength from strong (100 MPa) to very strong (>200 MPa) and are generally unweathered.

Rock cuts on ministry contracts are constructed with vertical walls using control blasting techniques such as preshearing and smooth wall blasting. Slopes in shale must be cut at 2 horizontal to 1 vertical to the bedding plane. Steeper slopes in shale of 1.5 horizontal to 1 vertical for example, suffer surficial failures, even when the slopes are well vegetated. Slopes at 1 horizontal to 1 vertical in shale undergo continuous deterioration, and vegetation cannot be maintained. Rock slope stability along Ontario highways is discussed in the document “Rock Cut Stability Along Ontario Highways” [8].

1.1.2 SOIL INVESTIGATIONS

The purpose of a soil investigation is to determine the existing geologic conditions and soil engineering properties which will influence roadway design and construction. The specific objectives of soil investigations are:

1. Determine the thickness of topsoil and organic strata;
2. Identify soil types and their engineering properties;
3. Determine the depth to bedrock, if relevant to design and construction;
4. Determine the groundwater conditions; and
5. Determine the existing pavement structure layer thicknesses and material properties.

This section provides a summary of the equipment and methods to fulfill the first four objectives. Apart from the requirement for coring to determine the thickness of the asphalt concrete, they can generally be applied to the fifth objective as well.

Investigation of soil and rock with respect to the support and stability of bridges, approach ramps, high embankments, high cut slopes, high rock cuts, large culverts or any situation requiring a more sophisticated geotechnical analysis are considered foundation investigations.

Additional information regarding specialized soil testing is presented in Section 1.1.4. The equipment used to carry out subsurface investigations can generally be divided into five groups:

1 .Vehicle-mounted power augers;
2. Manual augers;
3. Organic Samplers;
4. Backhoes; and
5. Core drilling equipment for rock.

Vehicle-Mounted Power Augers

Sterling augers are a commonly used vehicle mounted power auger. The sterling auger is equipped with a 200 mm diameter short-flight auger. The standard sterling auger can retrieve disturbed soil samples to a depth of 6 m, although auger extensions can be added to increase the depth of penetration to more than 15 m. The depth of augering may be limited if cobbly soil, cobbles/boulders, or when 'quick' conditions below the groundwater table are encountered.

Sterling auger's use is restricted to areas with firm surficial soil and relatively flat, unobstructed ground. Where poor or rugged ground conditions are encountered, track-mounted or truck mounted drills with continuous flight augers may be used. These drills are normally equipped with continuous flight solid and hollow stem augers and/or casing and wash boring equipment, and have the capability of carrying out split spoon and shelby tube sampling, as well as dynamic cone penetration tests and standard penetration testing.

Additional information on augering, split spoon and shelby tube sampling and on the standard penetration test are found in ASTM D-1452 [9], ASTM D-1586 [10] and ASTM D-1587 [11].

Manual Augers

Hand augering equipment is operated manually, although occasionally with motorized power heads. Manual augers may be equipped with short-flight augers, bucket augers, helical augers or different types of cutting bits. Manual augering methods for soil investigations are usually used when vehicle site access is restricted such as in soft organic deposits or when the investigation is limited to a surficial soil evaluation. Soil samples retrieved by manual auger are disturbed. The hand auger has a slow rate of advancement, and a limited depth of penetration (2 m in firm soils). Also, the reliability of determining soil stratigraphy is low if the depth goes beyond 0.5 m.

Organic Samplers

In order to obtain samples of organic silts, soft clay and peat, a hollow sampling device known as a peat sampler is used. The hollow device is made so that when it is pushed through the soil (organics) a solid internal cylinder remains in place. When the device is pushed to the top of the material to be sampled, the pipes are pulled up 300 mm. This withdraws the spring loaded internal cylinder and the sampler is then pushed into the soil 300 mm. The organic material is left to swell against the inside of the sampler before the pipe is rotated to shear off the bottom of the sample. The pipe and sampling device is then slowly withdrawn and the soil sample retrieved.

Backhoes

Track-mounted and truck-mounted backhoes are used to investigate soil conditions by open test pits. If test pits must be entered by testing staff, soil sampling beyond a depth of 1.2 m is generally not possible with the backhoe (due to safety concerns). If there is no need to enter the pit, the maximum depth of the test pit is controlled by the type of backhoe. Generally, the maximum depth of test pits with large backhoes is in the order of 6 to 7 m. Test pits are useful in evaluating the suitability of soil for earth borrow, as well as the suitability of granular deposits for processed aggregates. Test pits are sometimes used when large concentrations of boulders are anticipated.

The safe excavation and inspection of a test pit is covered under The Occupational Health and Safety Act (O.Reg.213/91). For all of the sampling procedures discussed previously, backfilling should be undertaken immediately upon the completion of sampling to prevent the risk of injury to animals or people.

Some advantages and disadvantages of the various soil sampling procedures are shown in Table 1.1.1. Additional information on sampling procedures is found in ASTM D-1452 [9], ASTM D-1586 [10] and ASTM D-1587 [11].

Table 1.1.1 Comparisons of Soil Sampling Procedures

Equipment	Advantage	Disadvantage
Sterling Auger	• Economic to operate • Fast • Can penetrate 6.0 m without auger extensions	• Access restricted to firm flat ground • Samples are disturbed • Sampling in wet, running sand difficult • Layer identification not as good as backhoe
Hand Auger	• Access to difficult areas • Low capital cost	• Limited depth • Slow • Samples are disturbed
Track or Truck-Mounted Auger	• Access to soft ground and rough terrain • Can penetrate more than 30m • Can penetrate below water table with hollow stem augers • Can retrieve moderately intact samples with special equipment	Stratigraphy definition not as good as backhoe
Backhoe (test pit)	• Good interpretation of stratigraphy and excavation stability • Can penetrate hard or cobbly soil	• Depth limited by the type of backhoe • Excavation in wet running soil not possible • Safety considerations

Core Drilling Equipment for Rock

Core drilling of rock may be required to assess rock type and degree of weathering from which excavation properties can be determined.

Sampling Depths and Locations

Soil tests and sampling should be carried out at frequent enough intervals to properly evaluate the properties of the soil strata, and to identify changes in soil conditions. Therefore, the spacing of test locations, and the sampling intervals are not always fixed and should be a function of the in-situ conditions. Section 3.6 should be referred to for further information.

Water Well Regulation 903

Regulation 903 is Ministry of Environment's water well regulation under the Ontario Water Resources Act. Although not technically water wells, boreholes advanced for soil investigation must comply with the Regulation depending on the depth of soil investigation. A geotechnical borehole is referred to as a test hole and is defined as a well that is made to test or to obtain information about ground water or an aquifer; and is not used as a source of water for agriculture or human consumption. Each test hole must be backfilled with commercially produced dry bentonite sealing material or other suitable sealant, or with clean, uncontaminated soil that has a grain size that is the same as or finer than the soil that was originally excavated. Test holes less than 3 m deep are exempt from the requirements of Regulation 903.

A licensed well technician is required to be present during test hole investigations. Professional engineers (P. Eng.) and Professional geoscientists (P. Geo.) are exempt from well technician licensing for some low risk work, but unless they are a direct employee of the government of Ontario, they must work for a firm with a well contractor's license.

1.1.3 SOIL TESTS

Laboratory testing is carried out on selected soil samples in order to determine engineering properties and performance characteristics, and to assess their impact on design and construction. The results of the tests also allow for an interpretation of the physical behaviour of the soil and may suggest ways by which the soils behaviour can be modified or improved. The soil profile is not normally included in the contract packages or generated by consultants, but the summary of borehole logs are provided with the contract drawings.

Tests of subgrade soils which are generally carried out following a soil investigation of a road alignment, together with the relevant laboratory test reference number from MTO's Laboratory Testing Manual [12], are as follows:

1. Particle size analysis, LS 702
2. Moisture content, LS 701
3. Atterberg Limits, LS 703 and LS 704
4. Moisture/Density Relationship (Proctor test), LS 706 and LS 707, if required, and
5. Methods for Calibration, LS-708, if required.

A general description and the purpose and frequency of the tests are described in the remainder of this section.

Particle Size Analysis, LS 702

Particle size analysis is a quantitative determination of the grain size distribution by mass of a soil. Grain size distribution of particles larger than 75 µm is determined by sieve analysis. Particles smaller than 2.0 mm are analyzed using the hydrometer. The results are expressed as a percentage of the mass of the soil sample which is more fine or coarse than a certain grain size.

Particle size analyses are carried out on representative samples of each soil type. The purpose of the test is to identify the soil types and gradations, and to assess frost-susceptibility. Soils with high silt contents are particularly susceptible to frost action.

Moisture Content, LS 701

The moisture content test is a determination of the ratio of the mass of water to the mass of dry soil, expressed as a percentage. The testing is carried out by weighing a soil sample before and after oven drying at 105°C for at least 16 hours.

The soil moisture content influences, among a number of other soil properties, the strength, plasticity and compressibility of a soil and is important in assessing the suitability of a soil for fill. The results are included on the borehole logs or in the contract drawings.

Atterberg Limits, LS 703 and LS 704

Atterberg Limit tests are index tests which determine the moisture contents of cohesive soils at the boundaries between the liquid state and the plastic state (the liquid limit), and between the plastic state and the semi-solid state (the plastic limit). The plasticity index is the numerical difference between the liquid limit and the plastic limit. The Atterberg Limit test results are frequently related to soil characteristics such as compressibility, strength, swelling potential and sensitivity (ratio of undisturbed to disturbed strength) of clay soils. The test results may be plotted on the ministry plasticity chart to assist in the classification of fine-grained soils.

Moisture / Density Relationship, LS 706 and LS 707

The moisture/density relationship, commonly referred to as the Proctor test, defines the relationship between the moisture content and the compacted dry density of a soil subjected to a given compactive effort. The test involves the compaction of soil into a 100 or 150 mm diameter mold by the action of a falling hammer. The soil is compacted into the mold at several different moisture contents and the density of the soil is measured at each of these moisture contents. Two tests are generally defined, which are described in LS 706 (the Standard Proctor) and LS 707 (the Modified Proctor). Soils in the Modified Proctor test are subjected to greater compactive effort or energy, than soils in the Standard Proctor test. The Modified Proctor is not routinely used by the ministry.

Proctor tests are used to define the optimum moisture content for compaction and the maximum dry density which can be achieved at the optimum moisture content. Research has indicated that there are strong inter-relationships between compacted density, moisture content and strength. Therefore, the Proctor dry density and moisture content relationship is used as a standard for comparison with field compaction results. Ministry specifications for the preparation of subgrade and granular materials are defined in terms of the maximum in-situ dry density to be achieved in comparison to the laboratory Proctor dry density.

California Bearing Ratio

The California Bearing Ratio (CBR) is an index which provides the relative bearing value (CBR) of base, subbase and subgrade materials. It compares the strength of a compacted soil to the strength of a standard crushed stone. The test involves the controlled penetration of a 50 mm diameter piston into the surface of a compacted soil sample to a maximum depth of 12.7 mm. The CBR of the soil is defined as a percentage of the force required to penetrate a standard crushed granular base. The force required to penetrate a standard crushed stone base material is given and the test results of other materials are given as a percentage of this standard. The CBR is used in some pavement design methods. The ministry does not routinely use the CBR test.

The CBR test is described in ASTM D-1883 [13].

Field Vane Test

The field vane test is used to determine the in-situ undrained shear strength of organic deposits and cohesive soils. The vane test consists of pushing a four-bladed vane with a height to diameter ratio approximately equal to 2 into the base of a borehole at selected depth intervals and measuring the torsional force required to cause a cylindrical surface formed by the outer edges of the blades to be sheared. Undisturbed and disturbed values are usually measured and used to calculate the sensitivity rating of the soil. The vane test is described in ASTM D-2573 [14]. It should be noted that for each vane, there is a vane constant which is used to convert the measured torsional force to a shear strength.

The undrained shear strength determined from the results of the vane test are used to assess the stability and compatibility of an organic or cohesive deposit subject to the load of an embankment. Additional information on the vane test is provided in reference document [15].

1.1.4 SPECIALIZED SOIL TESTING

The following are considered foundation investigations and should be carried out by specialized foundation engineers:

1. The investigation and recommendations for the design of bridge foundations and approach ramps;
2. The investigation and design of high earth and rock embankments, or deep cuts requiring stability analyses;
3. The investigation and recommendations for the design of large culverts;
4. Any project requiring retained soils systems; and
5. Any situation involving detailed bearing capacity, stability or settlement considerations.

1.1.5 LIME TREATED SUBGRADES

Lime treatment refers to the process by which the engineering properties of a subgrade soil, specifically its plasticity, strength, and compactibility, are improved by the addition of a small percentage of hydrated lime.

Lime treatment of subgrade soils in Ontario has the following objectives:

1. Reduce the plasticity of marginal soil to improve workability and thereby conserve the use of high quality materials.
2. Decrease the thickness of the pavement structure by increasing the strength of the subgrade.

When used in small quantities of 1 to 2% by weight of soil, lime modification decreases the plasticity, improves the compactibility and increases the strength of the soil by increasing the attractive forces between clay particles. This treatment is generally restricted to soft, wet, clay soils which would otherwise be wasted. When used in higher quantities of 5% or more, the procedure is referred to as lime stabilization with the result that soil cementation and strengthening occurs. Lime stabilization results in the formation of calcium silicates, similar to the process by which Hydraulic cement acts as the bonding agent in concrete.

Ministry experience with lime treatment has been confined to the addition of small quantities of lime to modify soft, wet, clay soils. In these cases, the lime has been spread on the surface of the subgrade soil and has been mixed into the soil to achieve a homogeneous mixture. The experience with a lime project in the Ottawa area involving both the modification and stabilization of highly sensitive and soft marine clay indicated that whereas the addition of small quantities of lime improved the strength and compactibility of the clay, it also rendered the clay frost-susceptible. The frost-susceptibility was attributed to the lime treatment causing an increase in the apparent grain size of the soil from predominantly clay to predominantly silt which has a higher frost susceptibility than other soil types. The addition of large quantities of lime (5% or more) substantially reduced the frost-susceptibility of the soil. Therefore, the use of lime modified or stabilized subgrade soil should only be recommended in conjunction with adequate laboratory freeze-thaw testing to establish the lime contents at which the soil will not become frost-susceptible.

Additional information on lime treatment is found in References [16] and [17].

1.2 GRANULAR MATERIALS

1.2.1 USE OF GRANULAR MATERIALS IN PAVEMENT STRUCTURES

Pavements are layered structures of selected and processed materials which have been designed to resist surficial wear, to support wheel loads, and provide drainage of water. There are numerous pavement materials which are in use in Ontario and some of these are described in Sections 1.3, 1.4 and 1.5. The various pavement types are outlined in Section 3.3.

The ministry currently specifies five types of granular materials which may be used for the construction and maintenance of pavement structures or embankments. These are:

1. Granular A;
2. Granular B, Types I, II and III;
3. Granular M (maintenance);
4. Granular O; and
5. Select subgrade material (SSM)

The physical and gradation specifications for Granular A, B, M, O and SSM are described in OPSS 1010 and Special Provisions, Part E of CDED Manual [18]. The physical requirements are shown on Table 1.2.1 and the grain size distributions of the materials are shown on Table 1.2.2. The significance of the specifications of all five materials are discussed in the following section.

Table 1.2.1 Physical Property Requirements

Laboratory Test	MTO Test Number	Granular O	Granular A	Granular B Type I Type III	Granular B Type II	Granular M	Select Subgrade Material
Freeze-Thaw Loss, % Maximum	LS-614	15	-	-	-	-	-
Fine Aggregate Petrographic Requirement	LS-616 LS-709	(Note 1)					
Micro-Deval Abrasion Coarse Aggregate loss, % maximum	LS-618	21	25	30 (Note 2)	30	25	30 (Note 2)
Micro-Deval Abrasion Fine Aggregate loss, % maximum	LS-619	25	30	35	35	30	-
Amount of Contamination	LS-630	(Note 3)					
Plastic Fines	LS-631	NP					
Plasticity Index, max.	LS-704	0	0	0	0	0	0

Note 1: For materials north of the French/Mattawa Rivers only: for materials with > 5.0 % passing the 75 µm sieve, the amount of mica retained on the 75 µm sieve (passing 150 µm sieve) shall not exceed 10 % of the material in that sieve fraction unless testing (LS-709) determines permeability values > 1.0 x 10-4 cm/s and/or field experience show satisfactory performance (prior data demonstrating compliance with this requirement will be acceptable provided such testing has been done within the past five years and field performance has been satisfactory.)

Note 2: The coarse aggregate micro-Deval abrasion loss test requirement will be waived if the material has more than 80% passing the 4.75 mm sieve.

Note 3: Granular A, B Type I, B Type III, or M may contain up to 15 percent by mass crushed glass and/or ceramic material. Granular A, O, B Type I, B Type III, and M shall not contain more than 1.0 percent by mass of wood, clay brick and/or gypsum and/or gypsum wall board or plaster. Granular B Type II and SSM shall not contain more than 0.1 percent by mass of wood.

Table 1.2.2 Production Requirements

Lab Test	MTO Test Number	Granular						SSM
		O	A	B (Note 1)			M	
	LS-602 (sieve)			Type I (Note 2)	Type II	Type III (Note 2)		
Sieve Analysis, % passing	150 mm	-	-	100	-	100	-	100
	106 mm	-	-	-	100	-	-	-
	37.5 mm	100	-	-	-	-	-	-
	26.5 mm	95.0-100	100	50.0-100	50.0-100	50.0-100	-	50.0-100
	19.0 mm	80.0-95.0	85.0-100 (87.0-100) Note 3	-	-		100	-
	13.2 mm	60.0-80.0	65.0-90.0 (75.0-95.0) Note 3	-	-		75.0-95.0	-
	9.5 mm	50.0-70.0	50.0-73.0 (60.0-83.0) Note 3	-	-	32.0-100	55.0-80.0	-
	4.75 mm	20.0-45.0	35.0-55.0 (40.0-60.0) Note 3	20.0-100	20.0-55.0	20.0-90.0	35.0-55.0	20.0-100
	1.18 mm	0 –15.0	15.0-40.0	10.0-100	10.0-40.0	10.0-60.0	15.0-40.0	10.0-100
	300 µm	-	5.0-22.0	2.0-65.0	5.0-22.0	2.0-35.0	5.0-22.0	5.0-95.0
	150 µm	-	-		-		-	2.0-65.0
	75 µm	0 -5.0	2.0-8.0 (2.0-10.0) Note 4	0-8.0 (0-10.0 Note 4)	0-10.0	0-8.0 (0-10.0) Note 4	2.0-8.0 (2.0-10.0) Note 4	0-25.0
Percent Crushed, minimum	LS-607	100	60	-	100	-	60	-
2 or more crushed faces, minimum, %	LS-617	85	-	-	-	-	-	-
% Asphalt Coated Particles, Coarse Agg, max.	LS-621	0	30	30	0	30	30	0

Note 1: Where Granular B is used for granular backfill for pipe subdrains, 100 percent of the material shall pass the 37.5 mm sieve.
Note 2: Where RAP is blended with Granular B Type I or Type III, 100 percent of the RAP shall pass the 75 mm sieve. Conditions in Note 1 supersede this requirement.
Note 3: Where the aggregate is obtained from an iron blast furnace slag source.
Note 4: Where the aggregate is obtained from a quarry or blast furnace slag or nickel slag source.

The typical uses and characteristics of the granular materials are described in the remainder of this section. For additional information, referred to CDED Manual [18]; and ministry Directive C-167 (SSM) [[19].

Granular A

Granular A is a well-graded material with 100% of the particles passing the 26.5 mm sieve size, and having at least 50% crushed particles, and no greater than 8% material passing the 75 μm sieve size (10% is allowed for rock quarry or slag sources) as shown in Figure 1.2.1. Owing to its relative fineness, well-graded distribution of particle size, and high crushed fraction, Granular A can be compacted to a dense stable material that can be fine-graded. Granular A is specified as base course, and shouldering material, as well as surface course for some sideroads.

Granular A is derived from crushed quarry rock, natural deposits of sand and gravel, iron blast furnace or nickel slag, or reclaimed hydraulic (Portland) cement concrete. RAP content is permitted as specified in MTO Special Provision SP 110S13, May 2010. Stee1 slag aggregates are acceptable in unpaved shoulders only due to the potential for expansion under certain moisture and temperature conditions.

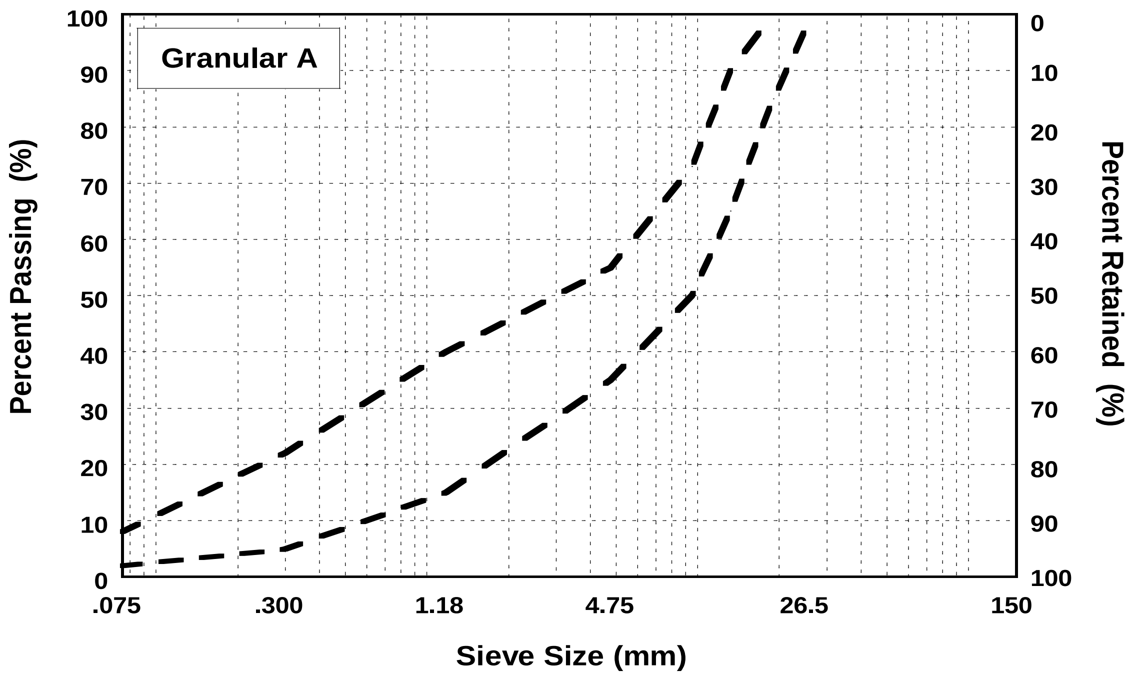


Figure 1.2.1 Grain Size Distribution Envelope of Granular A

Granular B

Granular B is a material with 100% of the particles passing the 150 mm sieve and no greater than 8% passing the 75 µm sieve size (10% is allowed for quarry sources).

Three types of Granular B are specified by the ministry:

Type I - can be uncrushed or crushed material

Type II - crushed bedrock

Type III – can be uncrushed or crushed material

Granular B Type I and Type III are derived from any sand and gravel deposit, talus rock, iron blast furnace or blended nickel slag, reclaimed hydraulic (Portland) cement concrete and may include RAP as prescribed in MTO Special Provision SP 110S13, May 2010. Quarried bedrock and iron blast furnace and nickel slag may be used for Type II. Recycled materials are not permitted.

Granular B Type I and Type III are generally coarser, may or may not include crushed particles and have a wider gradation. Consequently, they are susceptible to segregation and are more difficult to fine grade, and are not as stable as Granular A due to the absence of crushed particles. Granular B Type I generally has a finer gradation than Granular B Type III. Granular B Type III is used to achieve proper grading without the fine sands. Granular B Type II is a crushed material with a tighter gradation. Therefore, the performance characteristics are similar to Granular A, although due to its coarse gradation it cannot be fine graded. All Granular B types have satisfactory drainage properties. Granular B is specified as a granular subbase material. Granular B Type II is specified where Type I is not available and/or where the use of a higher quality subbase is required in the pavement structure.

See Figures 1.2.2, 1.2.3, 1.2.4 for grain size distribution curves for Granular B Type I, II and III.

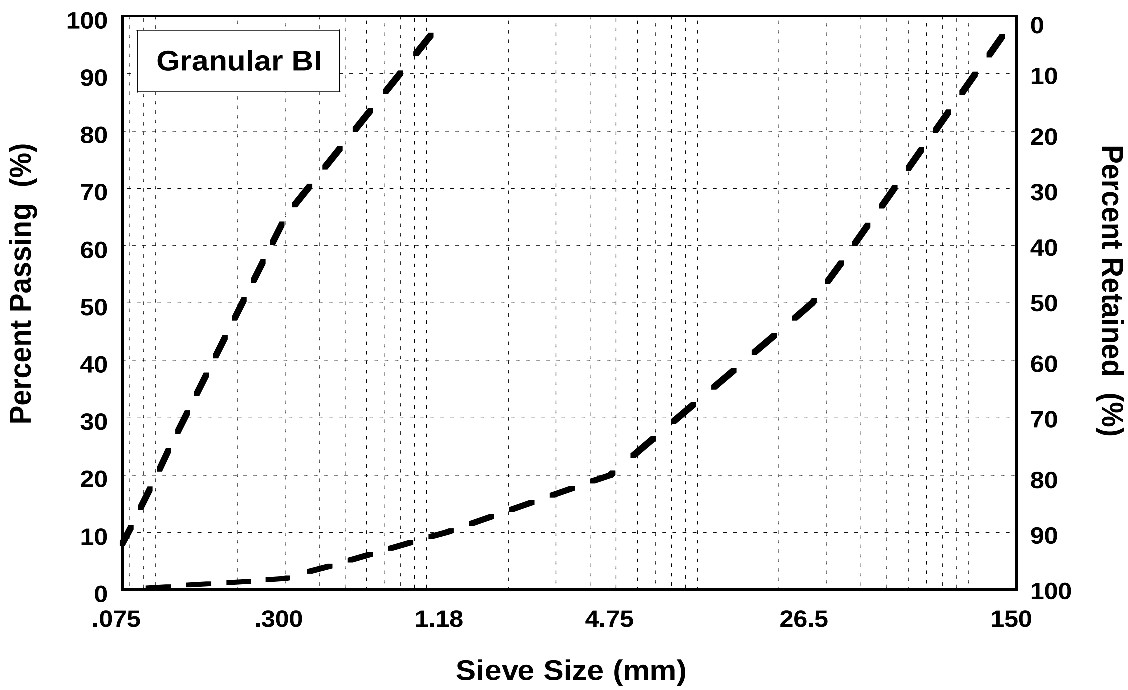


Figure 1.2.2 Grain Size Distribution Envelope of Granular B Type I

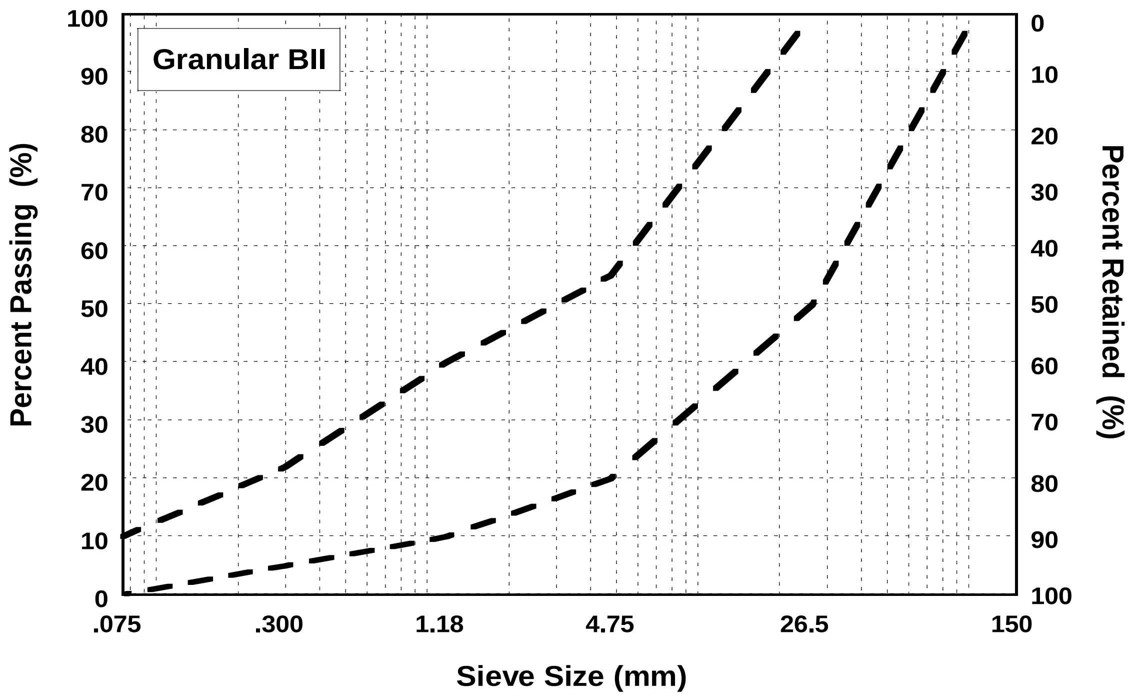


Figure 1.2.3 Grain Size Distribution Envelope of Granular B Type II

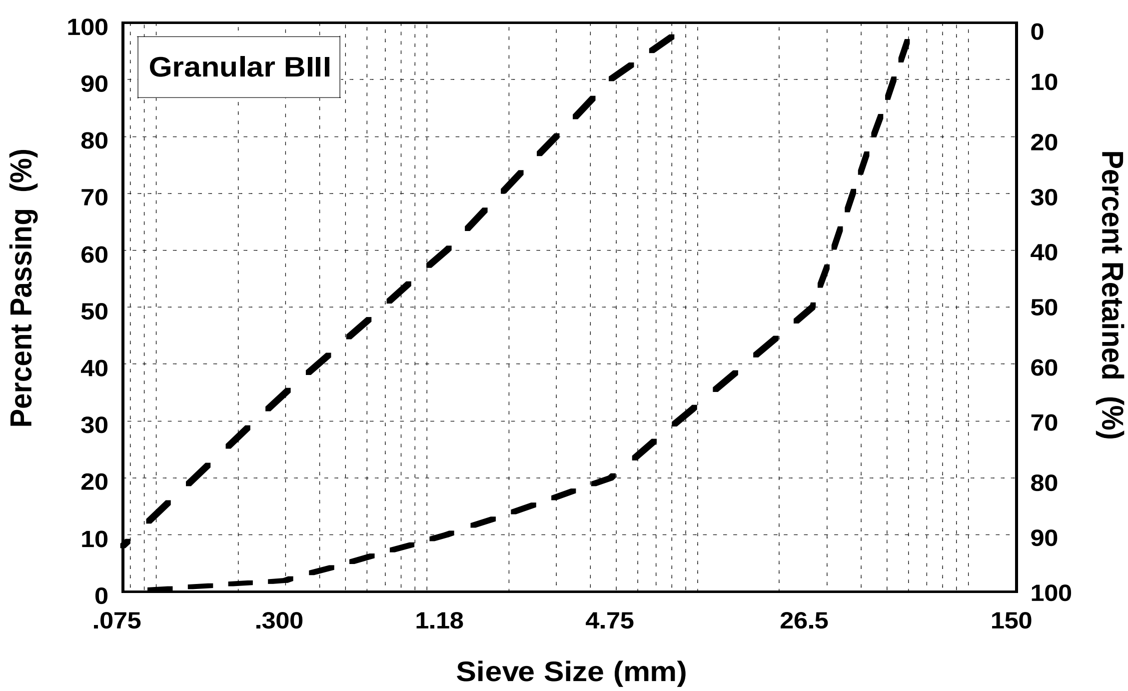


Figure 1.2.4 Grain Size Distribution Envelope of Granular B Type III

Granular M

Granular M is well-graded material with a maximum particle size of 19 mm and consists of at least 50% crushed particles (Figure 1.2.5). The physical properties and material sources of Granular M are identical to those of Granular A, although the gradation is finer than Granular A. Granular M has a low susceptibility to segregation and can be fine-graded for shoulder application and granular road maintenance material. It is also suitable as a surface course on granular roads and as a surface dressing material for fine-grading prior to paving. Granular M is sometimes used as a surface treatment aggregate, see Section 1.4.

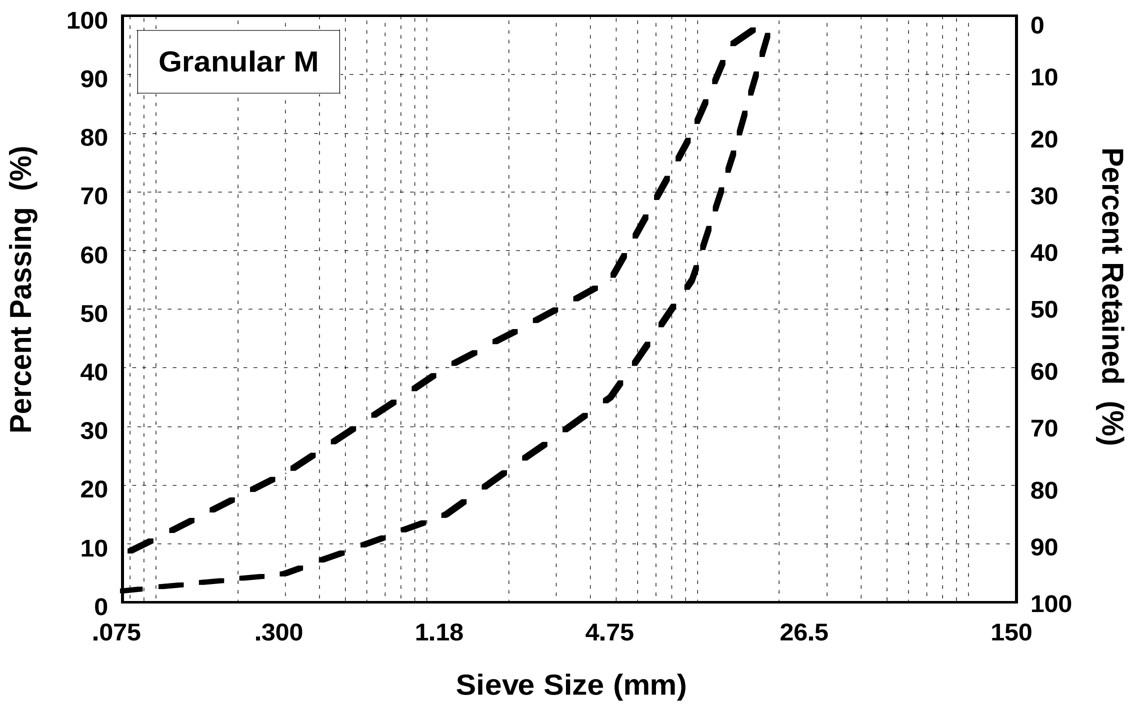


Figure 1.2.5 Grain Size Distribution Envelope of Granular M

Granular O

Granular O is a well-graded material with 100% of the particles passing the 37.5 mm sieve size, and no greater than 5% material passing the 75 µm sieve size (Figure 1.2.6). Granular O generally has a coarser distribution than Granular A. It is produced from crushed bedrock or crushed cobbles or boulders retained on the 50 mm sieve. Recycled or reclaimed materials are not permitted. Owing to its relative fineness, well graded distribution of particle size, and high crushed fraction, Granular O can be compacted to a dense stable material that can be fine-graded. Granular O is specified as base course, and shouldering material, as well as surface course for some sideroads. Also, used as a drainage layer on freeway projects below a concrete base.

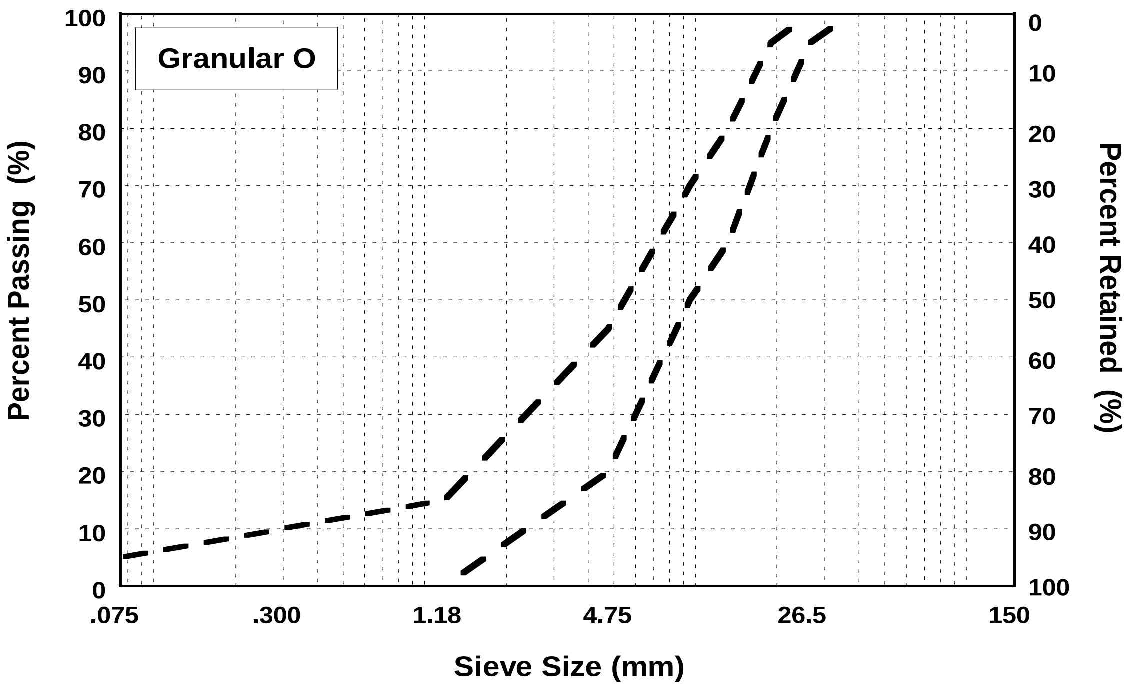


Figure 1.2.6 Grain Size Distribution Envelope of Granular O

Select Subgrade Material (SSM)

SSM is typically a sand, to sand and gravel sized material with a maximum particle size of 150 mm (Figure 1.2.7). SSM is non-plastic and recycled or reclaimed materials of any type are not permitted. Its non-plastic requirement is particularly important where SSM is to be used as swamp backfill or for any underwater placement where the use of uncompacted plastic material would lead to long-term settlement and possible failure. SSM is generally a good quality fill, although it has lower performance characteristics than Granular A, B, M and O. It is specified for use as backfill below groundwater level, generally to replace excavated organic materials or where a permeable fill is required to provide drainage. In addition, it may be specified on projects where local fine sands are readily available to achieve uniformity of subgrade performance and eliminate the need for more expensive imported subbase materials.

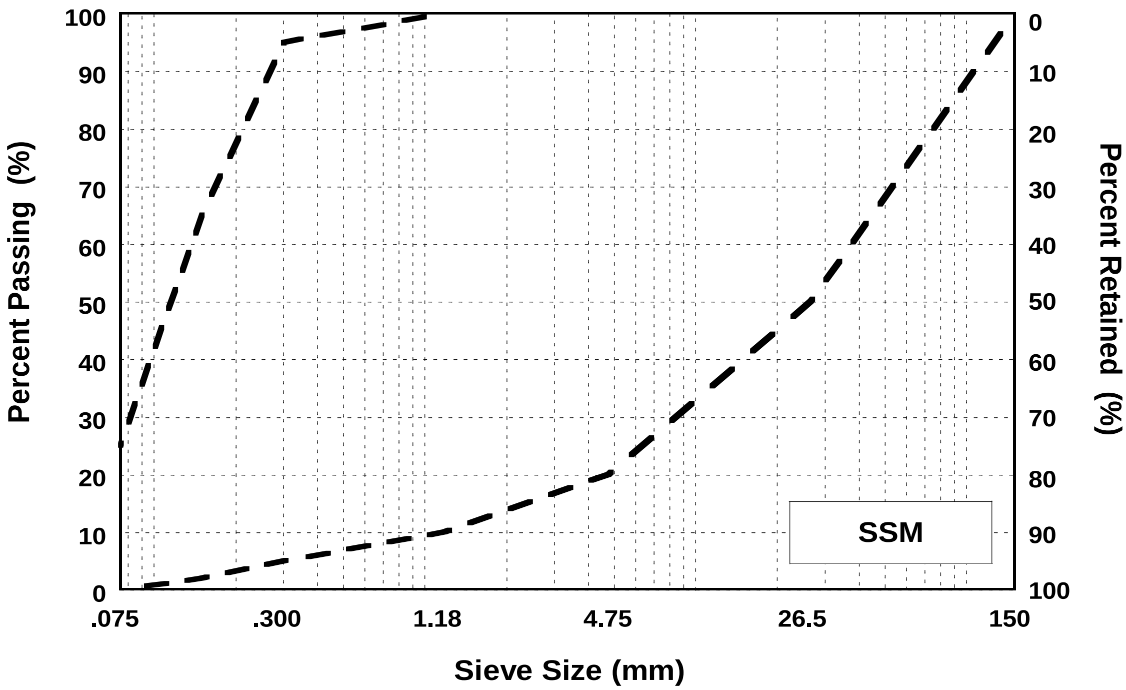


Figure 1.2.7 Grain Size Distribution of Select Subgrade Material

1.2.2 PRE-CONTRACT GRANULAR MATERIAL SAMPLING

Sources of granular materials used on ministry projects must be on the Aggregate Sources List (ASL) or must be sources identified by the contractor that meets the requirements of the specification. The most important aspect of sampling is that the collected materials accurately represent the source in terms of both quality and quantity. Sampling and testing requirements for production are also described in "Guidelines for Sampling of Granular Material for Acceptance Purposes" [20].

Additional information on sampling procedures is found in ASTM D-75 [21] and CSA A23.2-1A [22].

1.2.3 PRE-CONTRACT GRANULAR MATERIAL TESTING

Tests which must be carried out in order to verify that a granular material meets the ministry specifications are as follows:

1. Sieve Analysis of Aggregate, LS 602;
2. Micro-Deval (LS-618 and LS 619)
3. Petrographic Analysis, LS 609;
4. Plasticity Index of Fines, LS 703 and LS 704; and LS 631
5. Percent Crushed, LS 607.

Sieve analysis of aggregate, LS 602

Sieve analysis of aggregate is carried out to determine whether the grain size distribution is in accordance with the Laboratory Testing Manual [12]. The major purposes of the gradation specification are to ensure that the material meets the required standards of non frost-susceptibility, drainage, strength, and workability (segregation and fine grading). A washed gradation analysis is carried out to determine the quantity of fines in aggregates.

Micro-Deval , LS 618 and LS 619

The Micro-Deval test is a simple, inexpensive and precise test with a demonstrated correlation to field performance that measures abrasion resistance and durability of coarse and fine aggregates (MTO tests LS-618, LS-619). The test method allows for different aggregate gradings to accommodate a variety of specifications. For coarse aggregate, a 1500 g pre-soaked sample is placed in a small steel drum along with 2 litres of water and 5 kg of 9.5 mm diameter steel balls. Depending on the aggregate nominal maximum size, the drum is rotated at 100 ± 5 rpm for up to two hours. The percentage mass of sample passing the 1.18 mm sieve is measured and recorded as the Micro-Deval loss. For fine aggregate, the same equipment is used but a smaller sample (prepared to a Friction Modulus of 2.8), with 750 mL water volume and 1250 g abrasive charge are applied. The drum is rotated for 15 minutes and the percentage mass passing the 75µm sieve is determined as the Micro-Deval loss.

Petrographic Analysis of Coarse Aggregate, LS 609

The Petrographic Analysis is a subjective, routine quality test for coarse aggregate used to assess soundness, durability, hardness and strength. Simple index tests are used to classify the individual aggregate particles into quality types according to rock type, strength, hardness, and degree of weathering. Four quality categories are recognized: Good aggregate, (Factor 1), Fair aggregate (Factor 3), Poor aggregate (Factor 6) and Deleterious aggregate (Factor 10). A Petrographic Number (P.N.) is calculated by multiplying the percentages of each group by the appropriate factor. The products are then added up to arrive at the total P.N. It is emphasized that this test is based on laboratory studies and in-service performance for intended uses and prevailing conditions in Ontario.

The higher the P.N., the lower the quality of the aggregate. The weighting factors for certain categories of rock are different, depending on whether the aggregate will be used for asphaltic concrete, hydraulic (Portland) cement concrete, or granular materials.

Plasticity Index, LS 703 and LS 704 and LS-631

The Plasticity Index is described in Section 1.1.3, Atterberg Limits. The purpose of the test is to identify plastic fines which reduce strength in a granular material. For Granular A, B, M, O and SSM, the plasticity index must be zero.

Percent Crushed, LS 607

This test measures the percentage, by mass, of particles with at least one fractured face. The test is performed visually on the fraction of aggregate retained on the 4.75 mm sieve. The effect of crushed particles on a granular material is to promote interlocking between particles, thereby increasing the strength of the granular. The specified percent crushed is 60% for Granular A and M, 100% for Granular B Type II and Granular O.

1.3 TREATED GRANULAR BASES

Granular materials can be treated with a number of additives to enhance their engineering properties. The majority of the treatments are undertaken to improve granular strength and stability under loading. These treatments are most effective in areas where good quality aggregates are scarce. Granular materials are also treated with additives to control dust or to reduce settlement. Three major material groups of additives for granular treatment are:

- Hydraulic (Portland) Cement
- Bituminous Materials
- Others (Flyash, Calcium Chloride, Lignosulphonate)

1.3.1 HYDRAULIC (PORTLAND) CEMENT MODIFIED BASE AND SUBBASE

Cement treated base (CTB), lean concrete base (LCB) and roller compacted concrete (RCC) are terms used to describe three types of hydraulic cement treatment which have been used to improve the stability of granular materials. The ministry's experience with LCB has been relatively limited, since LCB and RCC are less susceptible to salt damage and weathering due to the higher cement content and lower permeability to water. In comparison, MTO has relatively more experience in CTB construction.

Cement Treated Base

Cement Treated Base (CTB) is constructed by adding about 5%, by weight, of hydraulic cement to a granular material and blending the two components to a uniform consistency at the optimum moisture content. The mixture is normally blended in a central plant, but sometimes the cement is spread on the surface and then mixed with a pulvimixer, roto-tiller, disc or similar equipment. The mixture is then compacted at the optimum moisture content. Blending and compaction at the optimum moisture content ensures that the CTB is compacted to a high density and that there is sufficient moisture for cement hydration to take place. Additional information on CTB is available from References [23],

[24], [25] and [26].

Lean Concrete Base

Lean Concrete Base (LCB) is a plant mixture of Granular A, water, and about 7% hydraulic cement and has the appearance and consistency of fresh hydraulic cement concrete. The 28-day compressive strength is in the range of 10 MPa. LCB has been used on a limited basis by the ministry in rigid pavement structures to replace CTB and Granular A which is subject to pumping action at joints.

Roller Compacted Concrete

Roller Compacted Concrete (RCC) is a zero slump mixture of hydraulic cement, water and quality aggregate with approximately 12 to 14% Hydraulic cement and 4% water by weight to initiate cement hydration. Prior to compaction it resembles a moist to very moist compactible aggregate. RCC is mixed in a central plant, placed and graded on the road surface, and compacted to a density of up to 2550 kg/m^3 using a heavy vibratory roller. Additional information on RCC is available in Reference [27].

Open Graded Drainage Layer

Open Graded Drainage Layer (OGDL) is a rapid draining layer located within the pavement structure that is between the overlaying concrete pavement, concrete base or hot mix asphalt (HMA) and the granular base course. The OGDL may be cement treated or asphalt treated. The aggregate shall consist of 100% crushed particles produced by crushing bedrock material with water-cement ratio of 0.37±0.01, porosity of 0.25 to 0.50 according to LS-627.

1.3.2 ASPHALT TREATED GRANULAR MATERIALS

Asphalt treatment of granular base materials improves the stability of the granular materials as well as providing increased resistance to the penetration of water. Asphalt treatments can include Bituminous Treated Base (BTB) and Emulsion Stabilized Base (ESB), asphalt treated OGDL and recently developed Expanded Asphalt Stabilization (EAS).

Bituminous Treated Base

Bituminous Treated Base (BTB), as used by the ministry, consists of Granular A mixed with 3 to 3.5% hot asphalt cement which is mixed in a central plant and placed with an asphalt spreader. In the short term, BTB provides an adequate wearing surface, however, due to its low asphalt cement content, it should be overlaid with asphaltic concrete to prevent raveling. Additional information on BTB is available from References [25] and [26].

Emulsion Stabilized Base

Emulsion Stabilized Base (ESB) is a central plant or road mix material consisting of slow to medium setting emulsified asphalt and granular material. ESB construction is carried out using a variety of equipment, including the Midland Paver, portable continuous mix plants, roto-tillers, pulvimixers and windrow mixing machines. Additional information on ESB is available from References [25] and [26].

Asphalt Treated Open Graded Drainage Layer

The asphalt treated OGDL shall consist of 100% crushed particles produced by crushing bedrock material. The percentage mass of asphalt cement in mixture shall be 1.8±0.2% according to OPSS 1150. The porosity of OGDL shall be 0.25 to 0.5 in accordance with LS-627.

Expanded Asphalt Stabilization

Expanded Asphalt Stabilization (EAS) is also called foam asphalt and it has been used by the ministry since 2001. EAS includes in-place full depth reclamation (FDR) of the existing hot mix asphalt (HMA) and underlying granular base. The reclaimed material is shaped, compacted and then stabilized in place by the addition of expanded asphalt. To expand the asphalt, a small amount of cold water is injected into hot asphalt cement (approximately 2.5% by mass) in the expansion chamber of a reclaimer/stabilizer. As the cold water turns to steam, the asphalt cement expands 10 to 15 times its original volume and is dispersed through nozzles onto the reclaimed material. Expanding the asphalt cement reduces the viscosity and increases adhering properties, facilitating mixing with cold, damp, reclaimed material. The expanded asphalt mixes readily with the fine aggregate particles, forming a mortar which bonds the coarse aggregate particles together. The stabilized material is then graded to the required profile and compacted. Following a minimum two-day curing period, the stabilized base is overlaid with HMA. This relatively new road base recycling method increases the strength of pavement and allows for a thinner overlay, while reusing existing materials and conserving aggregate and asphalt cement. EAS is specified in OPSS 331. Additional information on EAS is available from Reference [28].

1.3.3 LIME AND FLY ASH TREATED GRANULAR MATERIALS

Fly ash is a by-product of the combustion of coal in coal fired electric generating plants. From a chemical perspective, fly ash has been referred to as hydraulic cement without the lime (although some types of fly ash can have up to 25% lime). In the presence of water, fly ash combines with lime to form a strong cementitious material similar to hydraulic cement. A typical treatment consists of conditioning the granular material to the optimum moisture content, mixing with lime and fly ash in a central plant or in-situ, and placing and compacting the mix.

The hydration rate of the lime and fly ash mix is relatively slow compared to hydraulic cement treated granular, therefore the method is best suited to summer months when high temperatures favour an increased rate of hydration and strength gain. Drying of the mixture should be prevented during initial curing, and an asphalt primer should be applied to the road surface within 24 hours of compaction in order to prevent excessive moisture loss from surface evaporation.

The tendency for development of random cracks and deterioration caused by weathering and deicing salt are factors which have limited its use. Additional information on fly ash stabilized bases is contained in References [25], [26] and [29].

1.3.4 CALCIUM CHLORIDE

Calcium chloride acts to control dust, stabilize and improve compactibility when mixed into a granular road base. It is most frequently used by the ministry to control dust during construction and as part of on-going granular road maintenance programs. Calcium chloride is most effective in extending the performance of granular materials with a higher fines content rather than improving the performance of a clean, low fines content granular. Calcium chloride may be placed in liquid form from a tanker truck, in powder form by hand, or in larger quantities from a dump truck or other spreading device.

The principal property of calcium chloride responsible for its beneficial effects is its hygroscopic nature (tendency to absorb moisture from air). Water, which binds together the sand to clay size fraction of a granular material is held in the soil by the calcium chloride and therefore continues to be available long after the granular material has been compacted. The major drawback of the use of calcium chloride is its negative environmental impact when used near surface vegetation, surface water bodies, or close to the groundwater table. Additional information on calcium chloride stabilization is provided in References [25], [26] and [29].

1.3.5 MISCELLANEOUS GRANULAR TREATMENTS

Lignosulphonate

An alternative treatment which is used to control dust and to stabilize granular materials is lignosulphonate, which is a by-product of the pulp and paper industry. It is about 20% less efficient than calcium chloride; however its lower cost can make it an attractive alternative. Additional information on its use can be obtained from Reference [30].

Controlled Density Backfill (Unshrinkable Fill)

Controlled density backfill (unshrinkable fill), is a fluid mix of sand, aggregate and a low percentage of hydraulic cement. The design compressive strength of the mix is typically 0.4 MPa at 28 days. The mix can be produced in a ready mix concrete plant. The mix is used as trench backfill in areas where utility trench backfill settlement cannot be tolerated. Although the material cost of the controlled density fill is higher than that of granular backfill, the ease of placement, and the reduced maintenance costs over the long term can provide a cost-effective alternative to granular backfill. The use of controlled density backfill in trenches where frost-susceptible material is present is not recommended, since non-uniform road performance may result.

1.4 ASPHALTIC MATERIALS

Asphaltic materials described in this section include a range of plant and road manufactured products composed of asphalt cement binder and aggregate. Asphaltic material is used to surface most urban and arterial roads in Ontario and has the following properties:

1. Provides resistance to surface wear,
2. Reduces surface water infiltration;
3. Provides a smooth and rideable finish; and
4. Provides structural support to wheel loads.

There are two broad classes of asphalt pavement surfaces defined by the ministry:

Mix Systems	Hot, warm and cold mixed asphaltic concrete Hot and cold mixed stabilized base Recycled hot and cold mixed asphaltic concrete Travel plant mixes (cold) Slurry Seal Micro-surfacing
Layered Systems	Penetration Primer Surface Treatment Macadam Chip Seal

Road use, economic considerations, existing pavement condition and the relative functional characteristics of the pavement type determine which asphaltic material should be used to surface a road. The common types of asphalt pavement materials found in Ontario and their properties are described in Section 1.4.1.

The construction and material specifications for asphaltic materials used by the ministry are referenced in Table 1.4.1. OPSS references shown may be amended by special provisions for ministry use.

1.4.1 ASPHALT MATERIAL CATEGORIES

Hot Mix Asphaltic Concrete

Hot mix asphaltic concrete or hot mix asphalt (HMA) is a mixture of fine and coarse aggregate with asphalt cement which is mixed and placed in a heated condition. The components are heated and mixed in a central plant and placed on the road using an asphalt spreader. The HMA used by the ministry is designed, manufactured and placed within specified tolerances.

HMA can be categorized as either dense-graded or open-graded. The ministry mixes used to be designed using the Marshall method, but are now designed using the SuperPave system. Refer to Table 1.4.1 for the ministry specifications for asphalt pavement. Further design information for SMA and SuperPave is available in the MTO SuperPave and SMA Guide [43].

The types of HMA mixes are described in Reference [18] and are specified in OPSS 1151.

Table 1.4.1 Ministry Specifications for Asphaltic Pavements

TYPE	BINDER	AGGREGATE	MIX	CONSTRUCTION
SMA 19.0	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SMA 12.5	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SMA 9.5	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 37.5	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 25.0	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 19.0	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 12.5	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 12.5FC 1	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 12.5FC 2	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 9.5	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
SuperPave 4.75	OPSS 1101	OPSS 1103	OPSS 1151	OPSS.PROV 313
Hot Mix Recycled	OPSS 1101	OPSS 1103	OPSS 1150	OPSS.PROV 313
Cold Mix	OPSS 1103	OPSS 309	OPSS 309	OPSS 309
Surface Treatment	OPSS 1102 and 1103	OPSS 304	OPSS 304	OPSS 304
Primer	OPSS 1102	OPSS 302	OPSS 302	OPSS 302
Tack Coat	OPSS 1103	N.A.	N.A.	OPSS 308
Granular Sealing	OPSS 1102	N.A.	N.A.	OPSS 305

Note: OPSS can be found in:

http://www.raqsa.mto.gov.on.ca/techpubs/OPS.nsf/OPSHomepage

a) Stone Mastic Asphalt

Stone Mastic Asphalt (SMA) is a heavy duty gap graded hot mix asphalt with a relatively large proportion of stones and an additional amount of mastic-stabilized asphalt cement. The SMA mixture has an aggregate skeleton with coarse aggregate stone-on-stone contact to withstand damage due to heavy truck loads.

The additional amount of asphalt cement binder is required primarily to provide increased durability and resistance to aging and cracking of the mix. The use of durable aggregates and gradation provide superior rutting resistance. The stabilization of the extra asphalt cement and in particular, prevention of binder run-off during construction are achieved by: 1) an increase in fines and filler; 2) addition of organic or mineral fibre; 3) addition of polymer-modification; or 4) a combination of all three.

SMA designates hot mix types by the Nominal Maximum Aggregate Size, which represents the sieve size, in mm, through which at least 90% of the aggregates passes. There are currently three designations of SMA mixes as follows. Refer to Table 1.4.2 for details of the SMA mixes.

- SMA 19.0
- SMA 12.5
- SMA 9.5

b) SuperPave

SuperPave mix is HMA designed according to SuperPave criteria. SuperPave, which stands for Superior Performing Asphalt Pavements, was introduced in 1992 by the Strategic Highway Research Program (SHRP) under the sponsorship of the Federal Highway Administration of the United States. The SuperPave methodology incorporates a performance-based asphalt materials characterization system to improve the long-term pavement performance under diverse environmental conditions. It presently consists of the following elements:

- Asphalt cement specifications (now fully adopted in Ontario as Performance Grade Asphalt Cements (PGAC)).
- Revised aggregate specifications which include gradation control points, "consensus" properties such as fractured faces and clay content, as well as "agency" properties which are properties specified at the discretion of the agency.
- A new compaction method using the SuperPave Gyratory Compactor.

SuperPave designates hot mix types by the nominal maximum aggregate size, which represents the sieve size, in mm, through which at least 90 % of the aggregate passes. Currently, the following SuperPave mixes are specified. Refer to Table 1.4.2 for details of the SuperPave mixes.

- SuperPave 37.5
- SuperPave 25.0
- SuperPave 19.0
- SuperPave 12.5
- SuperPave 12.5FC 1
- SuperPave 12.5FC 2
- SuperPave 9.5
- SuperPave 4.75

A summary of mix uses and properties is provided in the following Tables 1.4.2 and 1.4.3:

Table 1.4.2 Summary of Hot Mix Asphalt

SMA 19.0	A premium binder course mix with enhanced rutting resistance for Traffic Category D and E roads.
SMA 12.5	A premium surface course with enhanced rutting resistance, water sprays reduction, and potential noise reduction for Traffic Category D and E roads. The most common SMA surface course type on Ontario highway is SMA 12.5.
SMA 9.5	A premium surface course with enhanced rutting resistance, water sprays reduction, and potential noise reduction for Traffic Category D and E roads.
SuperPave 37.5	Large stone binder course mix for the use when thicker binder lifts are required. To date this mix type has not been used in MTO projects.
SuperPave 25.0	Large stone binder course mix for use when thicker binder lifts are required.

SuperPave 19.0	Binder course mix typically used for all Traffic Categories (A, B, C, D and E). It replaces the previously used HL 4, HL 8, MDBC and HDBC mixes.
SuperPave 12.5	A surface course mix for Traffic Category B and C roads. It typically is used for all applications where premium rut and skid performance is not warranted. It is used on low volume highways, with the coarse-graded mix of <40% passing 2.36 mm sieve and fine-graded mix >40% passing 2.36 mm sieve. The aggregate source is selected at the contractor’s discretion. It replaces the previously used HL 3, HL 3 Fine and HL 4 mixes.
SuperPave 12.5FC 1	A surface course mix for Traffic Category C roads that provides superior rutting resistance and skid resistance through the use of premium coarse aggregate. It is used on medium traffic volume highways with the coarse-graded mix of <40% passing 2.36 mm sieve and fine-graded mix >40% passing 2.36 mm sieve. The coarse aggregate is selected from the pre-approved designated sources. It replaces the previously used HL 1 mix.
SuperPave 12.5FC 2	A surface course mix for Traffic Category D and E roads that provides superior rutting and skid resistance through the use of premium coarse and fine aggregates. It is used on high volume highways and freeways, with the coarse-graded mix of <40% passing 2.36 mm sieve and fine-graded mix >40% passing 2.36 mm sieve. The coarse and fine aggregates are selected from pre-approved designated sources. It replaces the previously used DFC mix.
SuperPave 9.5	A fine surface, padding, or leveling course mix for Traffic Category A and B roads and driveways.
SuperPave 4.75	A fine surface or leveling course mix used for miscellaneous applications.

Table 1.4.3 Aggregate Gradation for SMA and SuperPave Mixes

Hot Mix Asphalt Type	Percentage Passing by Dry Mass of Aggregates Sieve Size mm									
	50.0	37.5	25	19.0	12.5	9.5	4.75	2.36	1.18	0.075
Superpave 4.75	-	-	-	-	100	95-100	90-100	-	30-60	6-12
Superpave 9.5	-	-	-	-	100	90-100	32-90	32-67	-	2-10
Superpave 12.5, 12.5FC 1 and 12.5FC 2	-	-	-	100	90-100	45-90	45-55 (Note 1)	28-58	-	2-10
Superpave 19.0	-	-	100	90-100	23-90	-	-	23-49	-	2-8
Superpave 25.0	-	100	90-100	19-90	-	-	-	19-45	-	1-7
Superpave 37.5	100	90-100	15-90	-	-	-	-	15-41	-	0-6
SMA 9.5	-	-	-	-	100	70-95	30-50	20-30	(Note 2)	8-12
SMA 12.5	-	-	-	100	90-100	50-80	20-35	16-24		8-11
SMA 19.0	-	-	100	90-100	50-88	25-60	20-28	16-24		8-11

Notes:

1. Requirements for the 4.75 mm sieve are in addition to those normally used for Superpave.
2. For the SMA 9.5 mm the maximum percentage passing the 1.18 mm, 0.600 mm, and 0.300 mm sieves is 21, 18, and 15 respectively.

c) Warm Mix Asphaltic Concrete

Warm Mix Asphalt (WMA) is a technology that allows the mixing, transporting and lay-down process to take place at significantly lower temperatures (approximately 50ºC lower than traditional methods). Using less energy at lower temperatures during production results in up to a 50% drop in emissions. Other benefits of using warm asphalt include reduced exposure to fumes for workers during placement and compaction of the WMA.

The lower production temperature results in other potential WMA benefits include: facilitating a higher Reclaimed Asphalt Pavement (RAP) Content, decreased asphalt aging providing less potential for thermal cracking, improved compaction, improved joints, decreased thermal segregation and the ability to pave at lower ambient temperature, which can extend the paving season. An indirect benefit associated with WMA over HMA is the ability to haul the WMA over longer distances, since it is not required to maintain a high temperature.

Trial sections of WMA are operating successfully on MTO highways and are currently undergoing annual monitoring. For example, a WMA trial was carried out in 2008 on a section of Highway 15 from Smiths Falls northerly to Franktown. Further trials had taken place and results are being monitored.

Cold Mix Asphaltic Concrete

Cold mix is a mixture of emulsified asphalt and aggregate, produced and placed at ambient air temperatures. Liquid (cutback) asphalt is also used; however emulsified asphalt is normally specified due to its lower cost and because it is more environmentally friendly. The use of cold mix in Ontario is generally restricted to low volume rural roads, where HMA is not required.

Cold mix asphaltic concrete can be mixed in a central plant or in-situ on the road surface with a travelling mixer. Most of the ministry's experience with cold mix pavements has been with the Midland mix paver and road mix mulch pavements. The aggregate and emulsified asphalt are delivered to the Midland mix paver separately and the two components are blended together in a pug mill. Augers distribute the mixture to a screed which uniformly controls the thickness of the cold mix layer. Once the emulsified asphalt in the mixture surface starts to break, or set, the mixture is compacted. The performance of the Midland paver is described in Reference [31].

Cold mix materials are sub-divided into two categories:

1. Open-graded; and
2. Dense-graded.

Open-graded mixes are relatively stony mixes containing less than 10% fine aggregate.

Open graded mixes are more commonly used than dense-graded mixes due to difficulty in attaining uniform coating of the coarse aggregates in dense-graded mixes. Although open-graded mixes are known to be susceptible to ravelling, poor coating of the coarse aggregates can aggravate this condition. Open-graded mixes are surfaced with a light choke sand coating to seal the surface and to minimize aggregate pick-up by traffic. The mix surface is normally covered within a year with a single surface treatment (SST). Single surface treatments are described in the following section.

The ministry specifies five types of cold mix (OPSS 309), four of which are open-graded. Additional information on cold mix, construction techniques, and design methods are provided in References [31], [32] and [33].

Table 1.4.4 provides information on premium aggregate materials that may be used in the mixes. For surface course type, refer to MTO Surface Course Types Directive.

Table 1.4.4 Aggregate Source for Premium Hot Mix Asphalt

	SuperPave 12.5FC 1	SuperPave 12.5FC 2	SMA 12.5 or 9.5
Trap Rock	X	X	X
Meta-Gabbro	X	X	
Gneiss + Granite	X	X	
Dolomitic Sandstone	X	X	X
Quartzite	X		
Igneous Gravel	X		

Surface Treatment

Surface treatments consist of an application of emulsified or liquid asphalt and select aggregate over a prepared granular base or an existing surface. Following placement of the aggregate, the mixture is rolled and compacted to provide a driveable and dust-free surface. Emulsified asphalt is more commonly used than liquid asphalt due to its lower cost. This type of pavement surface is frequently used on light to medium volume roads which may or may not have an existing asphaltic concrete surface cover. Surface treatment of a granular road serves to control the infiltration of water, provide frictional resistance, improve ride quality, and control dust and stone pick-up. Surface treatments are also applied over existing surface treatment or asphaltic concrete pavements to restore frictional resistance and reduce the infiltration of water.

Surface Treatment Applications

There are different types of treatments that can be applied on granular surfaces and they are listed in the order most commonly used. Additional information on surface treatments is provided in Table 1.4.5 and on Aggregate Classes in OPSS 304.

(a) *Double Surface Treatment with Class 2 Aggregate, OPSS 304*

This surface treatment consists of two applications of binder (ie, emulsified asphalt) and Class 2 aggregate placed over a granular surface (the granular surface is not primed). The ministry prefers this application because it is durable, economical, can be completed in one day (no curing period), and is least subject to damage by rain and traffic.

(b) *Prime and Single Surface Treatment with Class 2 Aggregate, OPSS 302 and 304*

The treatment consists of an application of prime and Class 4 aggregate. Then, after 3 to 5 days curing period followed by a single application of binder and Class 2 aggregate.

(c) *Prime and Single Surface Treatment with Class 1 Aggregate, OPSS 302 and 304*

The treatment consists of an application of prime and Class 4 aggregate. Then, after 3 to 5 days curing period followed by a single application of binder and Class 1 aggregate. The ministry occasionally uses this application when a smooth, dust free surface is required.

(d) *Prime and Double Surface Treatment with Class 3 and 4 Aggregate, OPSS 302 and 304*

Treatment consists of an application of prime and Class 4 aggregate. Then after 3 to 5 days curing period followed by an application of binder and Class 3 aggregate, and a second application of binder and Class 4 aggregate. Prime and double surface treatment are used primarily by Municipalities.

(e) *Prime and Double Surface Treatment with Class 3 and 5 Aggregate, OPSS 302 and 304*

The treatment consists of an application of prime and Class 4 aggregate. Then after 3 to 5 days curing period followed by an application of binder and Class 3 aggregate, and an application of binder and Class 5 aggregate. This surface treatment option is commonly used by Municipalities.

(f) *Prime, OPSS 302*

An application of prime and Class 4 aggregate is placed over a granular surface. Primed surfaces are being surface treated and no longer used as riding surfaces or as dust layer because of the high cost and short (2 years) service life of the prime.

The following surface treatments are applied on existing bituminous surfaces, they are listed in the order most commonly used. Refer to the following list of surface treatment applications on bituminous surfaces.

(a) *Single Surface Treatment with Class 2 Aggregate, OPSS 304*

A single application of binder and Class 2 aggregate is placed over an existing bituminous surface that was either previously surface treated or less commonly, an asphaltic concrete pavement. The ministry uses this option frequently because it is economical, durable and Class 2 aggregate is readily available.

(b) *Single Surface Treatment with Class 1 Aggregate, OPSS 304*

Surface treatment with Class 1 aggregate is sometimes referred to as a chip seal. A single application of binder and Class 1 aggregate placed is over an existing bituminous pavement. The ministry occasionally uses this treatment when a smooth, dust free surface is required. It is more expensive than a single surface treatment with Class 2 aggregate because of a higher asphalt binder content and more costly aggregate.

(c) *Single Surface Treatment with Class 4 Aggregate, OPSS 304*

A single application of binder and Class 4 aggregate is placed over an existing bituminous pavement. It is used for situations where the existing pavement is ravelled, oxidized or porous. It should only be used when high frictional resistance is not required.

(d) *Single Surface Treatment with Class 5 Aggregate, OPSS 304*

A single application of binder and Class 5 aggregate is placed over an existing bituminous surface. This treatment option provides a fine-textured, dust-free surface.

Surface treatments are specified in OPSS 304. Recommended surface treatment types, and application rates based on traffic volume and existing road surface are described in Section B304 of the CDED Manual [18]. Additional information on surface treatments is provided References [34], [35] and [36].

Table 1.4.5 Surface Treatments

PRIME AND SURFACE TREATMENT APPLICATIONS FOR GRANUAR SURFACES		
Applications	**OPSS Specs**	**Remarks**
DST[1] with Class 2 Aggregates	304	Economical, durable, good skid resistance
P & SST[2] with Class 2 Aggregates	302 & 304	More expensive than DST
P & SST with Class 1 Aggregates	302 & 304	Used by Municipalities
P & DST[3] with Class 3 & 4 Aggregates	302 & 304	Used by Municipalities
P & DST with Class 3 & 5 Aggregates	302 & 304	Used by Municipalities
Prime	302	No longer used as a riding surface by MTO
Notes: 1 Double Surface Treatment 2 Prime and Single Surface Treatment 3 Prime and Double Surface Treatment		
SINGLE SURFACE TREATMENT APPLICATIONS FOR EXISTING BITUMINOUS SURFACES		
Applications	**OPSS Specs**	**Remarks**
SST[4,5] with Class 2 Aggregate	304	Economical, durable, good skid resistance
SST with Class 1 Aggregate	304	Smooth, dust-free surface
SST with Class 4 Aggregate[6]	304	For raveled, oxidized pavements
SST with Class 5 Aggregate	304	Fine-textured, dust-free surface
Notes: 4 Single Surface Treatment 5 Class 2 aggregates may cause dust problems in urban areas 6 Do not apply to flushed surface treatments, flushed pavements or when high friction values are required.		

Other Asphalt Surface Treatments

Other types of asphalt surface treatments include slurry seal, micro-surfacing, chip seal, fog seal, primer, tack coat and granular sealing. Apart from the primer, tack coat and granular sealing, these treatments are used by the ministry as pavement preservation treatments.

Brief descriptions of the treatments are given in the following:

Slurry Seal	Slurry seal is a mixture of emulsified asphalt, well graded fine aggregate and mineral filler (e.g., cement) applied in 3 to 6 mm layers. Slurry seal is applied over existing asphaltic concrete pavement and is used to seal the pavement and prevent ravelling. It provides no structural strength. Slurry seal is generally applied to the pavement surface from a travelling plant via a spreader box. Slurry seal is described in Reference [34]. This treatment is predominantly used by urban municipalities.
Micro-surfacing	Micro-Surfacing is a polymer modified asphalt emulsion mix consisting of aggregate, mineral filler, water and additive. It uses 100% crushed, high quality aggregate passing 9.5 mm. It is placed by specialized spreader equipment in lifts of 8 to 10 mm. Tack coats must be applied in most cases. It is effective in treating rutting problems. Microsurfacing is specified in OPSS 336.
Chip Seal	Chip seal is a rapid setting emulsion sprayed onto the pavement followed by rolling in the high quality, washed, crushed and single-sized aggregate typically 9.5 mm or 6.7 mm.
Fog Seal	Fog seal is a sprayed on application of low viscosity, slow to medium setting emulsified asphalt, which seals the pavement surface, controls water infiltration and controls oxidation. Some coating of surface aggregate may occur. Fog seal is discussed further in Reference [35]. This treatment is seldom used because of the resulting poor frictional characteristics.
Primer	Primer is an application of low viscosity liquid asphalt or emulsified asphalt used to seal a granular surface prior to the placement of surface treatment. Class 4 (sand) aggregate is (specified in OPSS 304) placed over the primer immediately upon application to permit the early return of traffic to the road. Primers are specified in OPSS 302 and are described in Reference [37].

Tack Coat	Tack coat is a diluted slow setting emulsified asphalt which is applied to existing pavement surfaces when specified. The placing of tack coats ensures good bonding between layers. Tack coats are specified in OPSS 308.
Granular Seal	Granular seal is a liquid or emulsified asphalt which is applied to the granular shoulder or rounding to prevent erosion. Granular sealing is specified in ministry specification OPSS 305.

Recycled Mixes

Recycled mixes are designed with reclaimed asphalt pavement (RAP) with additional virgin aggregate. Ministry's past experience with recycling has concentrated on hot mix recycling at central plants; however the technology for cold and hot in-place recycling is now well established and is briefly discussed in this section.

The specifications for the recycled hot mix product are identical to those of the virgin HMA, although softer grades of virgin asphalt cement are used to overcome the oxidation and hardening of the asphalt cement in the RAP. The design of the recycled mix is also influenced by the method by which the RAP was removed from the road surface. If the RAP is removed by cold milling, the cutting action of the teeth results in an increase in the aggregate fines content and a reduction in the stone content, as noted in the document "Cold Milling of Asphalt Pavement" [38]. However, adjustment in the operation of the milling machine and the use of split virgin coarse and fine aggregates instead of a single graded aggregate may alleviate most of the problems associated with aggregate degradation.

HMA recycling is discussed in more detail in the reference document "Asphalt Hot Mix Recycling" [39].

Reclaimed Asphalt Pavement and Roof Shingle Tabs

In OPSS 1151, ministry allows the use of Reclaimed Asphalt Pavement (RAP) and Roof Shingle Tabs (RST) in HMA pavement. Ministry allows up to 40% RAP incorporated into any hot mix asphalt, except for SMA, 0% RAP is allowed.

Roof Shingle Tabs (RST) produced by shredding up virgin shingle materials direct from the manufacturer. Ministry allows the use of RST to substitute RAP with the proportion of 1% RST equivalent to 10% RAP. For example, if the contract allows 20% RAP in the mix, the 20% RAP can be substituted for 2% RST or 10% RAP plus 1% RST. Although RAP cannot be used in SMA mixes, up to 3% RST is permitted in SMA mixes. Refer to OPSS 1151 for details.

Hot-In-Place Recycling

Hot recycled HMA mixes are generally made up of RAP, virgin aggregate, and asphalt cement binder. The proportioning of the mix depends on the characteristics of the RAP, the quantity of RAP available, the quality and availability of virgin aggregates, the type of plant and economic and environmental considerations. The RAP may be derived from an existing stockpile or directly from the road surface to be paved.

Hot in-place recycling (HIR) is used by the ministry for pavements that are generally free of major structural distress. Refer to specification OPSS 332 for details. The equipment available in Ontario offers two methods of HIR. In the first alternative, the existing surface is heated and scarified in-place, rejuvenated, reprofiled to a new grade, and then overlaid with a virgin mix. This operation is carried out with one machine in a single pass. The second alternative uses a two-stage process with the HMA overlay being placed separately with conventional paving equipment. Refer to Section 4.2.3 for more details.

Cold-In-Place Recycling (CIR)

CIR involves cold milling of the pavement surface and remixing with the addition of emulsified asphalt to improve the properties followed by screeding and compaction of the reprocessed material in one continuous operation. Refer to specification OPSS 333 for details. Additional information on cold in-place mix recycling is available in the reference document "Asphalt Cold Mix Recycling" [40] and Section 4.2.3 for of this Manual.

Cold-In-Place Recycling Expanded Asphalt Mix

Cold in-place recycling with expanded asphalt mix (CIREAM) is a recent development in CIR technology using expanded (foamed) asphalt rather than emulsified asphalt to bind the mix. Refer to specification OPSS 335 for details. The process is similar to CIR except that expanded asphalt is added to the mix instead of emulsified asphalt. Expanded asphalt is heated asphalt cement expanded from its normal volume by the addition of cool water. Detailed information on CIREAM recycling is available in a TRB paper on CIREAM [41] and Section 4.2.3 for of this Manual.

1.4.2 ASPHALT CEMENT, LIQUID ASPHALT AND EMULSIFIED ASPHALT

Asphalt cement is the binder used in all asphaltic concrete materials. In order to be placed on a road surface or mixed with aggregates, the asphalt cement must be in a liquid form. Asphalt cement is a thermoplastic material, therefore it may be liquefied by heating. Asphalt cement is also liquefied by dissolving it in a petroleum solvent, in which case it is termed liquid asphalt (or cutback) or by emulsifying it in water, in which case it is termed emulsified asphalt. Liquid asphalt and emulsified asphalt solidify when the solvent or water evaporates. It is sometimes desirable to improve the properties of the asphalt cement with additives. Additives are used to improve the flow, adhesion, oxidation resistance, or elasticity characteristics of the asphalt cement. A performance based grading specifications for asphalt binder was developed by the Strategic Highway Research Program (SHRP) to improve the physical properties of the asphalt cement to address thermal cracking of asphaltic concrete pavement.

The types, properties and uses of asphalt cements, liquid and emulsified asphalts, and additives used by the ministry are discussed in the remainder of this section. Additional information is available from Asphalt Institute reference document [42].

Asphalt Cement

Asphalt cement is a residue of crude oil distillation and is composed of asphaltenes, asphaltic resins and oily constituents. The characteristics of asphalt cement are largely dependent on the relative proportions of the three constituents.

Performance Graded Asphalt Cement (PGAC)

The Strategic Highway Research Program (SHRP) in the United States and the Canadian counterpart (C-SHRP) undertook the development of performance based grading specifications for asphalt cement as a key feature in the SuperPave system. SuperPave is a general term encompassing the methodology developed in the SHRP for selecting asphalt cement binders, estimating the fatigue, rutting, low temperature cracking and moisture damage performance of the asphalt concrete. Details of the SuperPave system are discussed in later sections.

A performance graded asphalt cement (PGAC) specification has been developed by the ministry (OPSS 1101) and it is now fully implemented by the ministry. Physical properties are specified and measured on the basis of binders that have been preconditioned using either a Rolling Thin File Oven (AASHTO T240) or a Pressurized Aging Vessel (AASHTO PP1).

Dynamic shear tests (AASHTO TP5) are performed on binder specimens that have been conditioned in a rolling thin film oven to ensure compliance with requirements that have been established to mitigate pavement rutting distress. The specimen also performed on a pressure aging vessel residue to ensure compliance with requirements that have been established to mitigate pavement fatigue distress.

Creep stiffness tests (AASHTO TP1) are performed on pressure aging vessel residue, to determine the flexural creep stiffness of binder specimens using a bending beam rheometer, to ensure compliance with requirement that have been established to mitigate low temperature pavement cracking distress.

PGAC is required to comply with specified requirements at both the low and high pavement service temperatures which must be determined on a project specific basis. For example, a binder identified as PG 64-28 must meet performance criteria at an average 7-day maximum pavement design temperature of 64ºC, and also at a minimum pavement design temperature of -28ºC.

PGAC is selected on the basis of the location of the contract; the type of hot mix (new versus recycled hot mix); and upgrades for highways with heavier traffic as appropriate. The province has been divided into 3 temperature zones for the purpose of selecting the appropriate PGAC for a project. Figure 1.4.1 and Table 1.4.6 provides the base performance grades for each zone. Two basic PGAC grades are specified for each zone, one for new hot mix or hot mix containing up to 20% RAP, and the other for mixes containing 21 to 40% RAP. Recycling ratios in excess of 40 percent should be addressed on a contract specific basis in consultation with the appropriate regional and head office units.

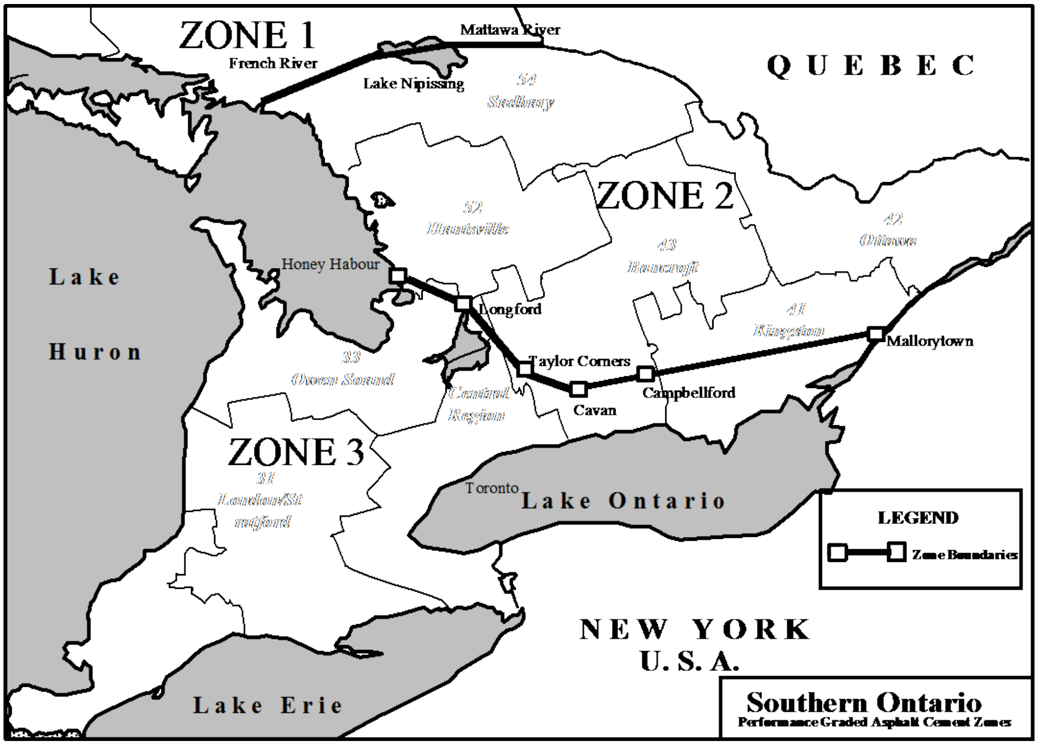


Figure 1.4.1PGAC Map for Southern Ontario

Table 1.4.6: Grade Selection for MTO Contracts

Mix Type	PGAC		
	Zone 1	Zone 2	Zone 3
New Hot Mix Or up to 20% RAP	52-34	58-34	58-28
21% to 40% RAP	52-40	52-40	58-34

For Ontario use, SuperPave guidelines have been interpreted in terms of highway classification and/or commercial truck traffic. It is recommended that the regional Geotechnical office be consulted for the application of these guidelines. For more details, refer to "MTO SuperPave and SMA Guide" [43]. Asphalt cements are specified in OPSS 1101 and listed in the DSM.

SuperPave recommends upgrades or bumps of the high temperature performance grade based on road classification and traffic loading. Current AASHTO Guidelines recommend a bump of one grade when the pavement carries slow moving traffic (slow is when the average traffic speed ranges from 20 to 70km/h) and a bump of two grades if the pavement carries standing traffic (when the average traffic speed is less than 20km/h). The guidelines are presented in Table 1.4.7.

Table 1.4.7: Guidelines for the Adjustment of PGAC High Temperature Grade Based on Roadway Classification and Traffic Conditions

Highway Type	Increase from Standard (Note 1)	Optional Additional Grade Increase (Note 2)
Urban Freeway	2 Grades	N/A
Rural Freeway **Urban Arterial**	1 Grade	1 Grade
Rural Arterial **Urban Collector**	Consider increasing by 1 grade if heavy truck traffic is greater than 20% of AADT	1 Grade
Rural Collector **Rural Local** **Urban/Suburban Collector**	No Change	1 or 2 Grades

Notes:
1. Upgrading of the high temperature grade is recommended for use in both surface and top binder courses, i.e., top 80 to 100 mm of hot mix.
2. Consideration should be given to an increase in the high temperature grade for roadways which experience a high percentage of heavy truck or bus traffic at slow operating speeds, frequent stops and starts, and historical concerns with instability rutting.

Liquid Asphalt

Liquid asphalt is a mixture of asphalt cement and petroleum solvent which is graded according to its rate of hardening and viscosity. Three grades of liquid asphalt are defined:

- Rapid Curing (RC)

- Medium Curing (MC)
- Slow Curing (SC)

The curing rate is dependent on the type of solvent used. Highly volatile gasoline type solvents are used for rapid curing mixtures, whereas kerosene and heavy distillates (diesel, gas, oil) are used in the medium and slow curing liquid asphalts. Rapid curing liquid asphalts are used as primers, whereas medium curing liquid asphalts are used in road mixes or patching material. Patching material which may be stored over long periods of time uses a slow curing liquid asphalt. The use of liquid asphalt cement is not as common as in past years due to the high cost of solvents. Liquid asphalts are specified in OPSS 1102 and are listed in the DSM.

Emulsified Asphalt

Emulsified asphalt is a mixture of asphalt cement and water in which microscopic beads of asphalt are suspended in water. An emulsifying agent is used to maintain desired stability of the emulsion. Emulsified asphalts are graded according to their setting time (note that for liquid asphalts curing time is used instead), viscosity (and for high floats, residual penetration) and electrical charge of the asphalt beads. Emulsified asphalts are sub-divided into three major types:

- Rapid Setting
- Medium Setting
- Slow Setting

They are further categorized according to whether the charge on the asphalt beads is negative (anionic) or positive (cationic). The choice of the emulsion type is a function of the aggregate type and gradation, and the charge on the surface of the aggregate. Experience has indicated that better coating of the aggregate is achieved when the asphalt beads and aggregate surface are of opposite charge. High float emulsions are better suited to grade aggregates with fines. Emulsified asphalts are specified in OPSS 1103, and designated sources are listed in the DSM.

Additives

Additives are used to improve the flow, adhesion, oxidation resistance or elasticity characteristics of an asphalt cement. Ministry use of additives has focused on the occasional application of anti-stripping additives to promote the adhesion of asphalt cement to aggregates. Stripping of asphalt cement from aggregates is generally related to excess moisture in the aggregate, aggregate type (some granites), dust coating, and the influence of water on the asphalt and aggregate bond. Increasing the asphalt content is also used to reduce stripping potential. Anti-stripping additives are generally categorized as hydrated lime or liquid anti-stripping [45]. Designated sources of liquid anti-stripping additives are provided in the DSM.

The use of polymer modifiers have been developed to reduce the temperature susceptibility of asphalt pavements. The modifiers are intended to reduce low temperature thermal cracking and improve resistance to deformation under vehicle loading at high temperatures. This is accomplished by improving the critical stiffness properties of the asphalt binder at both the low and high service temperature extremes.

1.4.3 AGGREGATES

Aggregates provide asphaltic concrete mixes with structural strength and frictional resistance. There are a number of different aggregates used by the ministry and the choice of a specific aggregate depends on the traffic type, volume and speed as well as on economic factors and performance history. Aggregates are categorized based on their grain size distribution and physical properties. The gradation and physical requirements of the coarse and fine aggregates are specified separately which allows for control of their quality and mix proportions. Aggregates used in cold mixes and surface treatments are not separated into fine and coarse fractions.

Aggregates used in many ministry asphaltic concrete mixes can be obtained from the sources listed in Table 1.4.4, provided that they meet minimum specified requirements determined by standard ministry tests. The standard tests which are used to determine the physical properties of coarse aggregates are as follows:

Micro-Deval , LS 618 and LS 619

This test is to demonstrate correlation to field performance that measures abrasion resistance and durability of coarse and fine aggregates. It is described in more detail in Section 1.2.3.

Magnesium Sulphate Soundness, LS 606

The soundness test is default for SuperPave 9.5 and 12.5 only, which simulates the effects of freeze-thaw cycles by subjecting the aggregates to cycles of soaking in a salt (magnesium sulphate) solution and subsequent drying. Salt is absorbed into the voids of the aggregates during soaking and crystallizes when the aggregates are dried. The growth and expansion of the crystals within the aggregate void causes internal stresses which simulate the freezing process. The degradation of the aggregate is a measure of the soundness of the aggregate. The gradation of the aggregate is determined before and after testing. Mass losses on specified sieves are accumulated in order to determine the mass loss percent of the entire sample. Fine aggregates are also assessed by this test.

Absorption, LS 604 and LS 605

This test measures the volume of aggregate pore space readily accessible to water. A dry sample of aggregate is placed in a water bath for 24 hours and the mass gain of the saturated surface dried aggregate is measured. In general, the greater the water absorption, the greater the potential for aggregate durability problems. The absorption test also provides an indication of the loss of asphalt cement into the aggregate.

Petrographic Analysis, LS 609

The petrographic analysis assesses the percentage and relative effect of deleterious substances in an aggregate. The test is described in more detail in Section 1.2.3.

Loss By Washing, LS 601

The degree of fine particle coating on coarse aggregate is determined by washing the aggregate on nested 4.75 and 0.075 mm sieves. The mass of fine particles passing the 75 µm sieve is a measure of the coating on the coarse aggregates. Coating of coarse aggregates may interfere with the adhesion of asphalt cement to aggregate.

Flat and Elongated Particles, LS 608

Aggregate particles with maximum dimensions exceeding their minimum dimension by more than a ratio of 4:1 are considered to be flat or elongated. The test is carried out visually on a representative sample of the aggregate. The stability of the mix may be detrimentally affected by elongated or flat particles.

Percentage Crushed, LS 607

This test measures the percentage by mass of coarse aggregate particles with at least one fractured face. Fractured faces improve the stability and strength of an aggregate by promoting interlocking between particles. This test is discussed in more detail in Section 1.2.3.

The minimum physical and gradation requirements of aggregates are provided in the ministry specifications which are referenced in Table 1.4.1.

1.4.4 HOT MIX ASPHALT DESIGN

The purpose of the mix design procedure is to determine the proportions of aggregates and asphalt cement binder such that the finished product meets the specified requirements. The ministry ensures that the mix meets performance requirements by specifying minimum standards (refer to Table 1.4.1 for references) based on pavement structure, traffic and environmental conditions.

SuperPave Method

The ministry adopted the SuperPave design method in 2002. SuperPave objective is to increase pavement life by reducing rutting, moisture sensitivity, fatigue and cold temperature cracking. There are two aspects of SuperPave:

- Implementation of SuperPave mixes including mix design and aggregate properties
- Implementation of Performance Graded Asphalt Cement (PGAC) binders

The SuperPave mix design method replaces the Marshall method used in Ontario and by most other jurisdictions. Laboratory compaction is achieved using the SuperPave Gyratory Compactor (SGC). The impact loading of the Marshall hammer in a 100 mm diameter mold has been replaced in the SGC with a kneading action achieved by rotating the base at an angle of gyration of 1.25° under a vertical ram pressure of 600 kPa. The use of a 150 mm diameter mold enables the use of larger top size aggregates. A key feature of the SGC is its capability to measure the height of the specimen as compaction proceeds. Simple calculations enable the height data to be converted to density and hence to percent compaction [43].

The SuperPave mix design procedure is described in details in “MTO SuperPave and SMA Guide” [43] and the reference publication [44]. Standard specifications OPSS 313 and 1149 have been revised by the ministry to reflect the newly adopted SuperPave mix design method.

1.5 HYDRAULIC (PORTLAND) CEMENT CONCRETE

1.5.1 CONCRETE PROPERTIES

Hydraulic (Portland) cement concrete is a mixture of sand and gravel or crushed stone particles bonded by a hardened paste of hydraulic cement and water. Hydraulic cement concrete mixes on ministry projects are designed by the contractor. This section contains a brief summary of properties important to the performance of concrete, and the relevant design parameters. Section 3.3.2 of this Manual describes the use of concrete pavement on ministry roads.

The characteristics of concrete which influence its placement and performance are:

Workability: The fresh plastic concrete must be workable such that it can be placed and consolidated without excessive segregation or bleeding.

Curing Rate: The concrete should harden at a sufficient rate to allow it to be placed into limited or full service at the earliest time. For new pavement construction the curing rate is normally several days. For overnight repair sections on existing roads, the decision to permit traffic is governed by compressive strength requirements. A minimum splitting tensile strength of 2.8 MPa at 10 days according to ASTM C-496186 is typical.

Strength: The compressive strength of concrete should be sufficient to support the design loads. For new concrete pavement/base, ministry uses cores 100 mm in diameter. For repairs and early strength determination, the ministry uses cylinders. Concrete pavements normally are specified to have a nominal design compressive strength of 30 MPa. Traffic can be allowed on the road when the concrete has a strength of 20MPa.

Durability: The concrete must be able to withstand the effects of freeze-thaw cycles and the service environment.

Volume Stability: The concrete must not experience severe volume change during curing or its service life in order to minimize the potential for cracking and deterioration. Properties of concrete which are of particular importance to pavement performance are durability and flexural strength. Concrete should be of sufficient durability to resist the effects of freeze-thaw, salt penetration, sulphate attack, and traffic abrasion. Flexural strength refers to the bending strength of the concrete required to support both the magnitude of the wheel loads and the number of load applications. These aspects of concrete behaviour and design are critical to concrete pavement performance and are discussed in more detail in the following sections.

Durability

Freeze - Thaw

The penetration of water and its subsequent expansion within the paste or the aggregates as the freezing point is approached can cause rapid deterioration of the concrete. Freeze-thaw effects can be reduced by using the lowest practical water to cement ratio, and by air entrainment. A low water to cement ratio renders the concrete less permeable, thereby reducing the penetration of water. Air entraining additives are mixed with the concrete to produce microscopic bubbles in the cement paste to accommodate water expansion during freezing. Air entrainment is a common requirement of concretes in Ontario and is required for all ministry concrete.

Salt Penetration

Salt penetrates the concrete in a solution form and as the freezing temperature is approached the liquid expands and high hydraulic pressures are set up. The internal expansive pressures are higher than those produced by salt-free water during freezing. Concrete designed with a low water to cement ratio, and air entrainment, should have suitable resistance to salt damage.

Sulphate Attack

Sulphate attack is a chemical deterioration of the concrete paste which occurs when sulphate, present in some soil, is dissolved in water and penetrates the concrete. The effect of sulphate attack is reduced with special sulphate resistant cements and a low water to cement ratio.

Traffic Abrasion

Concrete which is not overlaid with hot mix asphaltic concrete is subjected to abrasion and polishing from vehicle wheels. Abrasion resistance increases with strength, which in turn increases with decreasing water to cement ratio. The ministry specifies the use of 30 MPa compressive strength class concrete for concrete pavement and base, as outlined in OPSS 350. Abrasion resistance of this class of concrete has been found to be adequate. The use of sound, high quality aggregates is also necessary to provide abrasion resistance.

Flexural Strength

Concrete pavement or base should be designed to support flexural loads imposed by traffic. The flexural strength of concrete is determined during the design stage by two point loading of 150 x 150 x 900 mm beams prepared and tested in the laboratory according to ASTM C78-84. However, the splitting tensile test has been related to the flexural test, and due to practical constraints, it is frequently used instead of the flexural test to assess adequacy of the concrete for acceptance purposes. Ministry concrete pavements are normally designed with the requirements for compressive strength (30 MPa).

1.5.2 HYDRAULIC CEMENT

Hydraulic cement consists of a combination of clay and limestone materials which have been burned and ground to a powder-like consistency. When water is introduced, the cement chemically reacts with the water to produce a strong paste which gives hydraulic cement its high strength. The chemical reaction between the cement and water is referred to as hydration. The greater the degree of hydration, the higher the strength. The hydration process can continue for months or years after the concrete has been placed, although most of the significant strength gain occurs within a week to a month of placement. A significant by-product of the hydration process is heat, which can have a positive or negative effect on the performance of concrete, depending on the ambient temperature and the type of structure.

Different types of hydraulic cement are manufactured to meet the various physical and chemical requirements imposed by the project, the aggregates and the environment. The ministry commonly uses a GU (Type 10), Normal cement for all types of structures, including concrete pavements. It is a general purpose cement which is suitable on most projects where there are no special requirements. Other types of hydraulic cement used by the ministry are described in "Design and Control of Concrete Mixture" [46].

1.5.3 AGGREGATES

The primary functions of aggregates in concrete pavement are to provide strength to the mix, to resist shrinkage, and to resist abrasion where the concrete is used as a wearing surface. Aggregates occupy about 80% of the volume of the concrete, therefore their properties are important to the performance of the concrete. They should be hard, durable, strong, clean and free of chemicals or fine material which could affect hydration and the bond with the cement paste.

Properties of aggregates which are important to the performance of concrete are gradation, particle shape and texture, freeze-thaw resistance, abrasion resistance and chemical compatibility with the paste. These properties are briefly discussed in the following paragraphs.

Gradation

Gradation influences workability, cement and water requirements, shrinkage and durability. The primary objective when selecting the aggregate gradation should be to reduce the volume of voids to be filled by water and cement. This is accomplished when the aggregate is not deficient in any size fraction and has a smooth gradation curve. Aggregate which is graded in this manner generally provides the optimum properties in terms of workability, shrinkage and durability.

Particle Shape and Texture

Particle shape influences the workability and strength of the concrete, although its effect on workability is probably more significant. Rough angular particles require more water as well as cement. Cost and shrinkage increase as the cement content increases. Crushed particles promote improved bonding with the cement paste and consequently higher strength, although this is usually important only with high strength concrete or where flexural strength is required. In addition, the load transfer characteristics of undowelled joints are improved when crushed concrete aggregates are used. The ministry specifications for concrete always include dowel joints, and the specification does not have a requirement for crushed aggregates.

Freeze-Thaw Resistance

Freeze-thaw damage of concrete can take the form of pop-out of surface aggregates, or D-cracking when aggregates are saturated at the base of a concrete pavement. The freeze-thaw resistance of aggregates is related to porosity, permeability and tensile strength. The lower the porosity and permeability, and the higher the tensile strength, the less susceptible an aggregate is to freeze-thaw damage. Freeze-thaw resistance is also enhanced by minimizing the permeability of the cement paste (low water: cement ratio), which reduces the exposure of the aggregate to excessive moisture.

The susceptibility to freeze-thaw damage can be assessed from past performance, or the freeze-thaw testing of concrete specimens in the laboratory. The magnesium sulphate test (Section 1.4.3) is also used to assess the aggregate freeze-thaw resistance, although the test is not as reliable as actual freeze-thaw testing.

Wearing Resistance

The wearing resistance of a concrete pavement surface is related to the water / cement ratio of the paste, a lower w/c ratio having a greater resistance to abrasion. The aggregates also play an important part. Hard aggregates composed of such rocks as granites, and volcanic have a far greater wear-resistance than limestones and dolostones. Resistance to wear of aggregates is measured by the Aggregate Abrasion Value test, but it is not normally done. The abrasion resistance of hardened concrete is measured by any one of three ASTM tests (ASTM C418, 779, and 944). The frictional properties of the concrete pavement surface are not necessarily related to the wearing resistance of the concrete. The use of manufactured sand from a limestone quarry combined with a limestone coarse aggregate has been shown to result in a pavement with lower frictional resistance. If a limestone coarse aggregate is used, the fine aggregate must be a natural sand which contains significant amounts of very hard quartz particles.

Chemical Stability

Some aggregates are susceptible to attack from the alkali component (NaOH and KOH) in hydraulic cement. Aggregates subject to alkali reaction expand and generate internal tensile stresses which can result in significant deterioration of the concrete. Both carbonate and siliceous aggregates may be subject to alkali attack.

Aggregate types which have been identified as alkali reactive are mentioned in OPSS 1002. All concrete aggregate sources listed in the Concrete Aggregate Sources List (CASL) were either subjected to alkali reactivity tests or have a record of satisfactory field performance before they were placed on the list. The assessment is based on long-term material tests (ASTM C-227 and CSA A23.2-14A) in which the expansion of the material is measured over several months or years. Additional testing during the course of a contract may consist of a petrographic examination to confirm the consistency of the aggregate quality. Additional information on alkali reactivity is available in OPSS 1002 and References [46], [47] and [48].

The physical and gradation requirements for concrete aggregates are specified in OPSS 1001 and 1002 and generally address the factors discussed in this section. The physical requirements of the aggregates are assessed by the following tests:

- Material finer than 0.075 mm
- Micro-Deval
- Absorption
- Magnesium Sulphate Soundness (default)
- Flat and Elongated Particles
- Petrographic Number
- Freeze - Thaw of Coarse Aggregate LS-614

The significance of these tests for granular materials and asphaltic concrete is discussed in Sections 1.2.3 and 1.4.3 of this Manual and applies to hydraulic cement concrete aggregates as well.

1 5.4 MIXTURE DESIGN

The objective in designing a concrete mix is to produce a mix with acceptable workability of the fresh concrete, and strength, durability and good appearance of the hardened concrete. In addition to satisfying these performance criteria, the concrete must also be the most economical in consideration of the materials available, construction conditions etc. Economy is generally achieved by keeping the cement content to the minimum.

Although the mix design is the responsibility of the contractor, the ministry specifies:

1. the minimum 28 day curing strength;
2. the cement type;
3. the nominal maximum coarse aggregate size;
4. the tolerance and maximum value for slump;
5. the tolerance and maximum value of entrained air content; and
6. the minimum quantity of admixture.

Specification for concrete materials and productions are described in OPSS 350 and 1350 and in these reference publications [46] and [49].

Pervious Concrete

Pervious concrete pavements are being introduced as a storm water management best practice. A pervious concrete mixture contains little or no sand, creating a substantial void content. The pervious system consists of highly permeable, interconnected voids that drain quickly. Typically, between 15% and 25% voids are achieved in the hardened concrete, and flow rates for water through pervious concrete are typically around 0.34 cm/s. As such, it allows storm water to infiltrate directly, permitting a naturally occurring form of water treatment. Some data suggest that pervious pavement systems may contribute as much as 70 to 80% of annual rainfall to ground water recharge. In 2007, a pervious concrete pavement trial was constructed at the commuter parking lot located at Highway 401 and Guelph Line near Milton, Ontario. The final design consists of 240 mm of pervious concrete over 100 mm of open graded clear stone, over 200 mm of granular base material, over select subgrade material, over silty sand subgrade. Further trials had taken place and results are being monitored.

1.5.5 CEMENTITIOUS FLY ASH GROUT

Cementitious fly ash has been used by the ministry on an experimental basis to fill voids beneath concrete pavements. The subsealing is required to fill voids at joints under concrete pavements where fines have been lost due to pumping. In the past, remedial action consisted of overlaying the joints, however this repair method did not prevent the deflection of the slab and consequently progressive cracking and deterioration of the joints often occurred.

The performance requirements of the grout are that it be insoluble, incompressible, non-erodible, strong, flowable and unshrinkable. Fly ash hydraulic cement grouts, when compared to 100% hydraulic cement grout, have superior flow properties. Due to its relative fineness and spherical shape, the fly ash grout can penetrate very small voids and achieve greater subsealing than with a hydraulic cement grout. Several types of fly ash are available. In experiments carried out by the ministry, grout using Type C fly ash was found to be superior in terms of strength and shrinkage when compared to Type F fly ash. The mix used by the ministry in field trials consisted of 1 part cement, 3 parts Type C fly ash and 4.4 parts water.

Additional information on cementitious fly ash grout is available in the reference document [50].

1.5.6 MATERIALS FOR FAST-TRACK CONCRETE PAVEMENT REPAIRS

Fast-track (rapid setting) concrete mixes are used for pavement repairs to allow a quick restoration of the concrete road surface to traffic. Currently, repairs of high volume concrete roads are completed by removing the deteriorated hydraulic cement concrete, and replacing it with fast track concrete pavement mix.

Contractors are using calcium chloride based accelerators for the fast track concrete, and the ministry permits that as per the specification. The method should make it possible to attain strengths in excess of 20 MPa within 5 hours of placement of the concrete and therefore open the road to traffic with a minimum of delay. The attainment of the necessary high early strength is dependent on both the mix design and the construction methodology. Special features of the design include the addition of calcium chloride to concrete with a Type HE (Type 30), High Early Strength cement. During construction, temperature of 35°C shall not be exceeded at the time of placement, and insulation is required following initial set.

The ministry has developed a specification for fast-track concrete – SP399S43. Additional information on calcium chloride concrete is provided in the reference documents [51] and [52].

1.6 OTHER MATERIALS

1.6.1 LIGHTWEIGHT FILL

Lightweight fill may be used in the construction of high embankments over soft, compressible soil where the use of conventional fill would result in excessive settlements and global instability of the embankment. Lightweight materials are usually used in combination with other engineering solutions such as pre-loading, wick drains, and geogrid reinforcement to minimize long-term consolidation of roadway embankments. Situations where the use of lightweight fill may be considered include bridge approach embankments over organic or soft soils. More commonly used lightweight fill materials used by the ministry are air-cooled palletized blast furnace slag and extruded/expanded polystyrene (EPS).

Iron Blast Furnace Slag

Iron blast furnace slag is a lightweight by-product of the steel industry. The slag is 20 to 40% lighter than normal backfill as a result of voids within the processed slag particles. Pelletized blast furnace slag is the most commonly used lightweight fill material. The particles are rounded, less than 9.5mm in diameter and fairly uniformly graded. The material ranges from 11.5 to 14.0 kN/m^3 compared with 20 to 23 kN/m^3 for conventional fills.

Expanded or Expanded and Extruded Polystyrene

Expanded or expanded and extruded polystyrene has been used most frequently at approach embankments to bridges. For this application, the polystyrene is manufactured in rigid block form having a unit weight in the order of 0.5 kN/m^3.

The potential of using scrap rubber tire chips as a recyclable material is part of a provincial initiative to maximize waste reduction. Research study is currently underway to evaluate the feasibility of using shredded tires as embankment backfill material when coupled with geogrid reinforcement. Research of the physical and chemical characteristics of rubber tire chips and proposes a design method for construction of a trial embankment using a mixture of tire chips and soil is underway.

1.6.2 EXPANDED EXTRUDED POLYSTYRENE INSULATION

Expanded extruded polystyrene is a foamed, rigid board polystyrene designed specifically for in-ground applications. The extruded board has smooth, high-density skins and a dense closed-cell foam core.

Expanded extruded polystyrene is used to control differential frost heaving where there are significant changes in the frost-susceptibility of subgrade materials. For details, refer to Section 3.2.4.

The polystyrene insulation is available in various strengths, although normally 275, 415 and 690 kPa (40, 60 and 100 psi) compressive strength grades are used. A minimum cover of 300 mm of granular is generally sufficient to prevent damage due to construction vehicles. The thickness of the insulation boards range from 25 to 40 mm.

Additional information on expanded polystyrene insulation is provided in the publication [45], and Section 3.2.5 of this Manual. Installation is specified in OPSS 316 and sources of the insulation are listed in the DSM.

1.6.3 GEOTEXTILES

Geotextiles are synthetic fabrics which have a broad range of applications in the highway and earth related construction industry. Each application places a different demand on the geotextile, therefore the term geotextile has come to represent a large variety of products with a wide range of physical properties. Three broad categories of geotextiles are woven, non-woven, and knitted. Woven fabrics are manufactured from continuous lengths of plastic filaments which are woven together into a mesh like cloth. Non-woven fabrics are made up of random orientations of short to medium or continuous length filaments which are bonded together by thermal or mechanical means (needle punched). Knitted geotextiles are produced by knitting a continuous tube of geotextile specifically intended to wrap a perforated pipe.

Geogrids are available as an alternative to geotextiles for reinforcement applications. Geogrid is a coarse meshed polymer grid structure capable of developing high tensile stresses with little deformation. The ministry describes geotextiles as Class 1 or Class 2. Class 2 geotextiles are generally used where there is a requirement for strength. This is referenced in OPSS 1860.

In road construction, the uses of geotextiles may be subdivided into four main functions: filtration, drainage, separation and reinforcement. Approximately 95% of geotextiles in ministry applications are used as drainage and filtration application, and are non-woven.

Filtration

Filtering geotextiles act to restrict the migration of soil particles from a protected soil while allowing the free movement of water out of the soil. Typical applications are as a protective wrap around perforated pipes or French drains to prevent clogging, and under rip-rap to control the erosion of the underlying soil particles. An important property of a geotextile used as a filter is its pore size in comparison to the grain size of the soil being protected. The critical opening size of the geotextile is expressed as the Filtration Opening Size (F.O.S.). The relationship between soil grain size and F.O.S. to be used in design is described in the MTO Geotextile Design Manual [53]. Non-woven fabrics are normally used for filter applications.

Drainage

Geotextiles may be used as transport media for the collection and removal of excess water. Of particular importance in this application is the ability of the water to move through the plane of the fabric. For this reason it must have a high in-place hydraulic conductivity, adequate thickness to transmit the required volume of water, and sufficient compressive strength to prevent crushing. It should also be capable of filtering the drained soil to prevent clogging of the fabric and loss of fines. Non-woven fabrics are normally used in this application. Drainage type geotextiles may be used as vertical and horizontal drainage paths in embankments, under rip-rap, or behind retaining walls to prevent the build-up of hydrostatic pressure.

Separation

The separation process involves the physical separation of two dissimilar materials which would otherwise mix and result in a deterioration of strength or drainage properties. A separating geotextile must also act as a filter layer and often as a reinforcing layer. Typical applications include for placement between the subgrade soil and subbase to prevent pumping, between soft organic soils and embankment fill to prevent mixing or between rock fill and granular to prevent loss of the granular. Woven and non-woven geotextiles are used for this application.

Reinforcing Geotextile

A reinforcing geotextile acts to strengthen a soil by mobilization of the tensile strength of the geotextile. A typical use of the geotextile may be in a reinforced earth type of structure to form vertical walls or steep embankments. Alternatively, it may be used to improve the load carrying capacity of a pavement by spreading the wheel load over a larger area of subgrade. Woven geotextiles or geogrids are most commonly used in this application.

Additional information on geotextiles and the design requirements are provided in the reference documents [54] and [55]. OPSS 1860 outlines the material specifications and designated sources are listed in DSM.

REFERENCES TO CHAPTER 1

[1] Chapman, L.J. and Putnam D.F. 2007, *Physiography of Southern Ontario*, Ontario Geologic Survey, Miscellaneous Release – Data 228.

[2] *Ontario Soil Survey*, numerous maps and publications, Canada Department of Agriculture.

[3] *Ontario Geologic Survey*, numerous maps and publications, Ministry of Northern Development and Mines (formerly of Ministry of Natural Resources).

[4] Ontario Ministry of Transportation of Ontario, Engineering Materials Office, *Soil Classification Manual*, 1980.

[5] Robert G. Gorman, Chris A. Rogers, *Recommended Practice For Establishing Rock Elevation For New Highway Construction*, MERO report #30, January 2007.

[6] Franklin, J.A., *Evaluation of Shales for Construction Projects, An Ontario Shale Rating System*, Report RR-229, prepared for and published by Ontario Ministry of Transportation and Communications, 1983.

[7] McMaster, J.B., *Shale Fill Construction Practices*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-19, 1978.

[8] Magni, E.R., *Rock Cut Stability Along Ontario Highways, Proceedings from Fourth International Symposium on Landslides*, Toronto, Ontario, 1984.

[9] American Society of Testing and Material (ASTM) Standards D-1452, *Standard Practice for Soil Investigation and Sampling by Auger Borings*.

[10] American Society of Testing and Material (ASTM) Standards D-1586, Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

[11] American Society of Testing and Material (ASTM) Standards D-1587, *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes*.

[12] Ministry of Transportation of Ontario, Materials Engineering and Research Office, Highway Standard Branch, *Laboratory Testing Manual*.

[13] American Society of Testing and Material (ASTM) Standards D-1883, *Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils*.

[14] *American Society of Testing and Material (ASTM) Standards D-2573*, Standard Test Method for Field Vane Shear Test in Cohesive Soil.

[15] Northwod, R.P. and Sangrey, DA., *The Vane Test in Organic Soils*, Canadian Geotechnical Journal, Volume 8, Number 1, 1971.

[16] Batten, A.M. and Hanks, A.J., *Design, Construction and Frost-susceptibility of Lime Stabilized Marine Clay In Highway Subgrade Fill*, presented to the Transportation Research Board, Washington, 1983.

[17] Hanks, A.J., *Frost Heave Measurements of Lime Modified Soil from the Ottawa Area in the Laboratory*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-42.

[18] Ministry of Transportation of Ontario, Highway Design Office, *Contract Design, Estimating and Documentation Manual*.

[19] Ministry Directive C-167 (SSM)

[20] Ministry of Transportation and Communications, Engineering Materials Office, *Guidelines for Sampling of Granular Materials for Acceptance Purposes*, EM-67,1985.

[21] American Society of Testing and Material (ASTM) Standards D-75, *Standard Practice for Sampling Aggregates*.

[22] Canadian Standards Association (CSA) Standards A23.2-1A, *Sampling Aggregates for Use In Concrete*.

[23] Portland Cement Association, *Soil-Cement Construction Handbook*, EB003F.

[24] Portland Cement Association, *Soil-Cement Handbook*, EB052.

[25] Israel, R.W., *Soil Stabilization and Cold-Mix Recycling*, Bomag (U.S.A), 1210 Kenton Street, P.O. Box 959, Springfield, Ohio, 1982.

[26] Bomag (U.S.A), Bornaging A Road. *A Guide to Soil Stabilization and Cold In-Place Recycling*, Bomag (U.S.A), 1210 Kenton Street, P.O. Box 959, Springfield, Ohio, 1987.

[27] American Concrete Institute, *Roller Compacted Concrete*, ACI Committee Report Number AC1207.5 R - 80,1977.

[28] Lane, B., Akin, A., and Kazmierowski T., *Five-year Performance of Expanded Asphalt Stabilization on Highway 17, South of Wawa, Ontario*, Draft MTO MERO report, 2007.

[29] United States Department of Transportation, *Flyash Facts for Highway Engineers*, Federal Highway Administration Report FHWA-DP-59-8, 1986.

[30] Roads and Transportation Association of Canada, *Guidelines for Cost-Effective Use and Application of Dust Palliatives*, Ottawa, Canada, 1987.

[31] Corkill, J.T., *Construction and Performance of Cold Mix Placed by the Midland Mix Paver*, Ministry of Transportation and Communications, Engineering Materials Office, report EM-15, 1977.

[32] Aquin, R.C., *Construction and Performance of Emulsified Asphalt Cold Mix Pavements in Ontario*, Ministry of Transportation and Communications, Engineering Materials office, report EM-22, 1978.

[33] The Asphalt Institute, *Asphalt Cold-Mix Manual*, The Asphalt Institute Manual Series No. 14, Maryland, U.S.A.

[34] The Asphalt Institute, *A Basic Asphalt Emulsion Manual*, The Asphalt Institute Manual Series No. 19, Maryland, U.S.A.

[35] Cooper, J. and Aquin, R., *How to Achieve a Good Surface Treatment*, Ministry of Transportation and Communications, Engineering Materials Office report MI-57, 1984.

[36] Cooper, J.A. and Aquin, R., *Double Surface Treatment Without Prime Versus Prime and Single Surface Treatment, Interim Report*, Ministry of Transportation and Communications, Engineering Materials Office, report MI-60, 1984.

[37] Cooper, J.A. and Aquin, R., *How to Achieve A Good Priming Job*, Ministry of Transportation and Communications, Engineering Materials Office, report MI-59, 1984.

[38] Rogers, C.A. and Scott, R.J., *Cold Milling of Asphalt Pavement*, An Investigation of Aggregate Degradation, Ministry of Transportation and Communications, Engineering Materials Office, report EM-59, 1982.

[39] The Asphalt Institute, *Asphalt Hot-Mix Recycling*, The Asphalt Institute Manual Series No. 20, Maryland, U.S.A., 1981.

[40] The Asphalt Institute, *Asphalt Cold-Mix Recycling*, The Asphalt Institute Manual Series No. 21, Maryland, U.S.A., 1983.

[41] Lane, B., and Kazmierowski, T., *Implementation of Cold In-Place Recycling with Expanded Asphalt Technology in Canada*, Transportation Research Board, Journal No. 1905, TRB, National Research Council, Washington, D.C., 2005, pp. 17-24.

[42] The Asphalt Institute, *Introduction to Asphalt*, The Asphalt Institute Manual Series No. 5, 1982.

[43] MTO Bituminous Section, *MTO SuperPave and SMA Guide*, MERO, March 2008.

[44] Chan, S., *Development of a Methodology for Assessing the Technical and Economic Benefits of Pavement Laboratory Equipment*, A Master thesis presented to University of Waterloo, Waterloo, Ontario, 2001.

[45] Field, F., *Evaluation of Anti-Stripping Additives for Asphalt Concrete Mixes, Effectiveness and Recommended Usage*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-9, 1977.

[46] Portland Cement Association, *Design and Control of Concrete Mixtures*, available for download: http://www.cement.org/bookstore/profile.asp?id=155

[47] Rogers, C.A., *Evaluation of The Potential For Expansion and Cracking Due To Alkali-Carbonate Reaction,* Ministry of Transportation and Communications, Engineering Materials Office, report EM-75, 1985.

[48] Rogers, C.A., *Alkali Aggregate Reactions, Concrete Aggregate Testing and Problem Aggregates in Ontario,* A Review, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-31, 1985.

[49] American Concrete Institute, *Recommended Practice for Selecting Proportion for Normal and Heavy Weight Concrete*, AC 2 1 1.1-74.

[50] Kazmierowski, T. and Wrong, G., *Experimental Cement Grout Subsealing of a Concrete Pavement in Ontario*, Presented at the 65th Annual Meeting of the Road and Transportation Association of Canada Annual Conference, 1986.

[51] Bugler, J.W., *Use of Calcium Chloride Accelerated High-Early Strength Concrete for Slab Repair,* Presented at Transportation Research Board, Washington, D.C., U.S.A., 1986.

[52] Sturm, H. J. and Kazmierowski, T. J., et al, *"Fast Track" Concrete Pavement Repairs,Ontario's Experience*, Presented at 1989 RTAC Annual Conference, Calgary, 1989.

[53] *MTO Geotextile Design Manual*.

[54] Barsvary, A.K. and MacLean, *M., Engineering Geotextiles, Properties, Use Criteria, Tests and List of Acceptable Products*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-45, 1981.

[55] Barsvary, A.K. and Korgemagi, P., *The Geotechnical Use of Filter Fabrics in Highway Construction*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report EM-13, 1982.

BIBILOGRAPHY TO CHAPTER 1

Aquin, R.C., Korgemagi, P. and Lynch, D.F., *Surface Treatment Test Sections with Polymer-Modified Emulsified Asphalt*, Ontario Ministry of Transportation and Communications, Engineering Materials Office, report MI-116, 1988

McMaster, B. and Wrong, G., *The Role of Expanded Polystyrene in The Ontario Provincial Transportation System*, Presented at Transportation Research Board, Washington, D.C., U.S.A., 1987.

Ministry of Transportation of Ontario, Engineering Materials Office, *Soils Manual*, 1980.

Strategic Highway Research Program. 1994. *The SUPERPAVE Mix Design System Manual of Specifications, Test Methods, and Practices*. National Research Program: Washington D.C.

SUPERPAVE Vs. The Canadian Winter: Low Temperature Performance at C-SHRP Test Roads. C-SHRP Technical Brief # 19. June 2000.

Transportation Association of Canada, Pavement Design and Management Guide, TAC, Ottawa Canada, 1997.

(Blank Page)

APPENDIX A:

MTC
Soil Classification
Manual

(Blank Page)

MTC

SOIL CLASSIFICATION MANUAL

Ministry of
Transportation and
Communications
Ontario

ENGINEERING MATERIALS OFFICE
DOWNSVIEW
JANUARY 1980

NOTES:

The M.T.C. Soil Classification Manual supersedes the draft of the "M.T.C. Soil Classification System, published by the Engineering Materials Office in January, 1979.

FOREWORD

Upon the recommendation of the Regional Geotechnical Heads it was decided that the Unified Soil Classification System, with some modifications, should be adopted by this Ministry. This new policy was announced in Ministry Directive B-13, Provincial Roads, dated 1978 11 06.

Accordingly, the Manager of the Engineering Materials Office appointed a Task Force to prepare a manual for the uniform usage of this modified system which is now known as "The MTC Soil Classification System".

This publication, prepared by the Task Force, contains a detailed description of the MTC Soil Classification System and explains the procedures for classifying soils, both visually in the field and by laboratory methods.

This classification system shall be used by Ministry staff for the identification and description of soils in the field, based on visual examination and simple manual tests. This system shall also be used in delineating particle size characteristics, liquid limit, and plasticity index, when precise classification is required.

The description of soil layers in borelogs, Soils Design Reports, Foundation Investigation and Design Reports, etc., shall be carried out using the soil terminology and group symbols of the new system.

Suggestions and queries pertaining to the MTC Soil Classification System should be addressed to any member of the Task Force. The Task Force members are:

A.K. Barsvary Engineering Materials Office (Chairman)
M. Devata Engineering Materials Office
A.J. Hanks Engineering Materials Office -
S. Chen Eastern Region
S.G. Wilson Northern Region

TABLE OF CONTENTS

GLOSSARY

Aggregate	Particles of natural rock or artificial solid materials which satisfy the gradation and quality requirements of the type of material specified.
Clay	Fine grained soil having particles smaller than 75 µm that can be made to exhibit plasticity (putty like property) within a range of water contents, and which exhibits considerable strength- when dry.
Coarse-Grained Soils	Soils with more than half of the particles between 75 mm and 75 µm.
Cobbles	Rock fragments, usually rounded or subrounded, with an average dimension between 75 mm and 200 mm.
Cohesionless Soil	A soil that, when unconfined and air-dried, has little or no strength, and that has little or no cohesion when submerged.
Cohesive Soil	A soil that, when unconfined and air-dried, has considerable strength, and that has significant cohesion when submerged.
Dilatancy	The expansion of cohesionless soils when subjected to shearing deformation.
Earth	Earth and soil are synonymous, but the term "earth" is also used as a pay item in contract documents to distinguish any excavation or fill material from rock or granular as specified in MTC Specification Form 200.
Effective Diameter (D10)	Particle diameter corresponding to 10% finer on the grain-size curve.
Fine-Grained Soils	Soils in which more than half of the particles finer than 75 µm are in the silt and clay range.
Fines	Portion of a soil finer than 75 µm.
Glacial Till	Material deposited by glaciations, usually composed of a wide range of particle sizes, which has not been subjected to the sorting action of water.

Gradation	(Grain Size Distribution). Proportion of material of each grain-size present in a given soil.
Grain Size Analysis	(Mechanical Analysis). The process of determining gradation.
Granular (Soil, Material)	(a) Synonymous to cohesionless soil or material. (b) Coarse-grained soils from which base and sub-bas aggregates can be produced.
Granular A and 16 mm and Crushed Granular A and B	Mixtures of sand and crushed rock, gravel, or slag with specified gradation bands complying with the requirements of MTC Specification Form 1010, usually used for base courses or shoulders.
Granular B & C	Sands and/or gravels of various proportions (may contain traces of cobbles), having specified gradation bands and complying with the requirements of MTC Specification Form 101 0, usually used for sub-base course aggregates.
Granular D	Crushed rock screenings with a gradation band specified in MTC Specification Form 1010, used as an alternate subbase material.
Gravel	Rounded; subrounded or angular particles of rock that will pass a 75 mm sieve and be retained on a 4.75 mm sieve.
Ground Water	Water that is free to move through a soil mass under the influence of gravity.
Ground Water Level	Elevation at which the pressure in the water is zero with respect to the atmospheric pressure.
Liquid Limit - (w_L)	(a) The water content corresponding to the arbitrary boundary between the liquid and. plastic states of consistency af a soil. (b) The water content at which .a pat of soil, parted by a groove of standard dimensions, will flow together for a distance of 13 mm under the impact of 25 blows in a standard liquid limit apparatus.
Muck	An organic soil of soft to very soft consistency
Optimum Moisture Content (w_{opt})	The water content at which a soil can be compacted to a maximum dry density by a given compactive effort.

Organic Soil	Soil with a high organic content, may contain shells and/or fibres. In general, organic soils are very compressible and have poor load sustaining properties.
Overburden	(a) Earth overlying granular deposits which must be removed prior to extraction of the granular deposit. (b) Earth or other unconsolidated materials, either transported or formed in place, overlying bedrock.
Parent Material	(a) Material from which a soil has been derived. (b) The unconsolidated material, unaltered by weathering, from which the overlying 'A' and 'B' horizons developed.
Peat	A mass of organic matter usually fibrous in texture in various stages of decomposition, generally dark brown to black in colour and of spongy consistency.
Plasticity	The property of a soil which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change.

INTRODUCTION

The term soil or earth, in engineering practice, is defined as sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, and which may or may not contain organic matter. Only hard rock, which remains firm after exposure, is excluded from the definition.

To the engineer or technician engaged in the design and construction of roads, foundations and earthwork, the physical properties of soil, such as density, permeability, strength, compressibility and interaction with water, are of primary importance. It is apparent that a standard system of describing the soil and placing it into a category or group which has distinct engineering properties is most desirable in order to provide a common language for the transmission of information and experience. The description of soil usually gives detailed information on the gradation, plasticity, colour, particle characteristics, moisture content, strength and the like. Few if any soils will have identical descriptions. Soil classification on the other hand, places a soil in one of a limited number of groups, on the basis of a few key characteristics. These characteristics must be easy to measure and must be significant in indicating the engineering properties and performance of the particular soil.

Such a classification system was originally proposed by Professor A. Casagrande in 1942 for the U.S. Corps of Engineers to be used in airfield construction. In 1952, after some modification, this system was adopted by the U.S. Bureau of Reclamation and was named the "Unified Soil Classification System". The advantages of the Unified System in identifying soils on the basis of their engineering properties were soon recognized and as a result it gained wide acceptance all over the world. Casagrande later suggested an extension to his classification and some of the users also made slight modifications to suit their needs. The principles of the original system however, remain the same. The MTC Soil Classification System introduced in this publication is basically the Unified System modified by Casagrande with some further slight changes implemented by the Task Force including particles larger than 75 mm and metric conversion.

Chapter 1

THE CLASSIFICATION SYSTEM

(1.1) Boulders and Cobbles

Particles larger than 75 mm are excluded from the Unified Soil Classification System. However, the amount of such material is of great importance in Ministry practice, hence any description of soils should always contain information on these particles. They are classified into boulders with an average dimension larger than 200 mm and cobbles with particles between 200 mm and 75 mm.

Because of the large particle sizes, the percentage and dimensions of these materials can only be adequately described by visual observations of excavations or exposures.

If more than half of a mixed coarse grained stratum is larger than 75 mm the deposit shall be called boulders and/or cobbles. The portion of the material smaller than 75 mm shall be described by the constituent soils according to the rules of the MTC Classification System. (See Section 1.3 and 1.4.)

> **Example:** A material consisting of 60% cobbles, 32% gravel and 8% sand may be described thus: Gravelly cobbles, trace of coarse sand.

If the material contains less than 50% boulders and/or cobbles, the "oversize" particles should be removed and the classification should be carried out on the portion of the material smaller than 75 mm, according to the rules of the MTC Classification System. The description of the soil should contain information on constituent particles larger than 75 mm.

> **Example:** Soil in a test hole, consisting of approximately 45% gravel, 50% sand and 5% silt and having a few randomly distributed cobbles may be described thus: Well graded gravelly sand, trace of silt, a few random rounded cobbles of 100 mm to 200 mm size throughout the deposit.

(1.2) Soil Categories

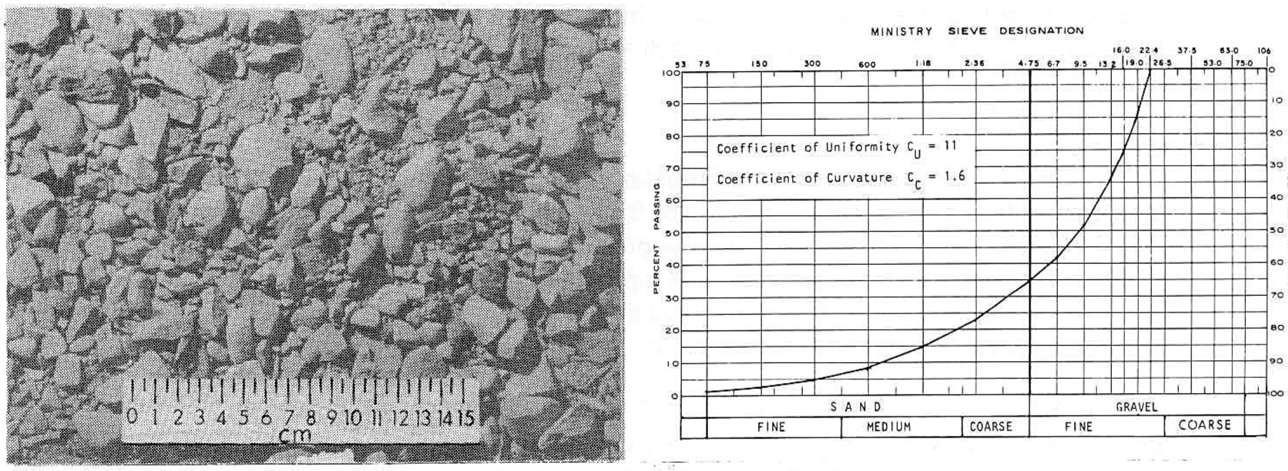
The system is based upon the sizes of the particles smaller than 75 mm, the distribution of the particle sizes, and the properties of the fine-grained portion. First, the soils are divided into three major categories: (1) coarse-grained soils, (2) fine-grained soils, and (3) highly organic soils. Second, the soil is subdivided by either gradation or plasticity characteristics.

Coarse-grained soil (sand and gravel) is that material which has particle sizes between 75 mm and 75 µm. The smallest size in this category is about the smallest particle size which can be distinguished with the naked eye.

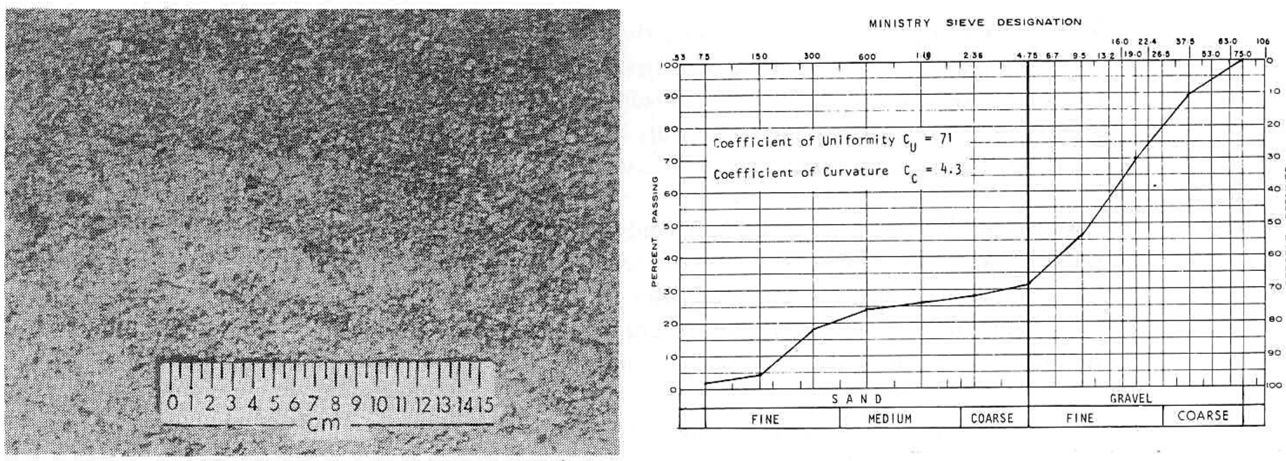
Photographs of typical coarse-grained soils and the corresponding grain size curves are shown in Figure 1.

Fine-grained soil (silt and clay) is that material having particle sizes smaller than 75 µm.

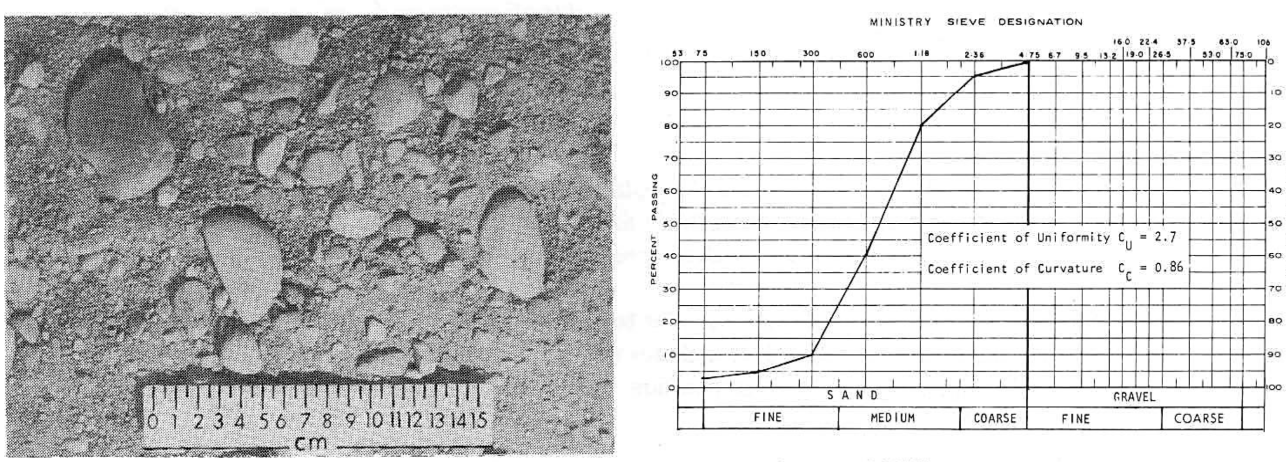
Highly organic soils are peat or other soils which contain substantial amounts of organic matter.



(a) Well-graded sandy gravel (GW)



(b) Poorly-graded coarse to fine (uniform) sand (SP)



(c) Poorly-graded (gap-graded) sandy gravel (GP)

In the MTC System, soils having 50% or more material larger than 75 µm are classified as coarse-grained while those having less than 50% are classified as fine-grained. No laboratory criteria are used for the highly organic soils, but generally they can be identified in the field by their distinctive colour and odour and by their spongy feel and fibrous texture.

(1.3) Coarse-Grained Soils

The two major divisions of coarse-grained soils are gravel and sand. A coarse-grained soil having more than 50% of the coarse-grained fraction (larger than 75 µm) retained on a 4.75 mm sieve is classified as gravel, denoted by the symbol G. A coarse-grained soil having more than 50% of the coarse-grained fraction passing a 4.75 mm sieve is classified as sand, denoted by the symbol S. Coarse-grained soils are further sub-divided either by their gradation (distribution of grain sizes) or by the properties of the fine-grained fraction of the soil.

The classification and criteria for each group are given in the MTC Classification Chart in the Appendix, together with a Plasticity Chart which is instrumental in classification by this System.

(a) Less Than 5% Pass the 75 µm Sieve

Those coarse-grained soils having less than 5% passing the 75 µm sieve are subdivided by their gradation and are given the classification of GW, SW, GP and SP meaning, respectively, Gravel - Well-graded, Sand - Well-graded, Gravel - Poorly-graded, and Sand – Poorly graded. These groups include those soils in which the fine-grained portion is so small that it should not affect engineering characteristics.

GW Group: Well-graded gravels and sandy gravels which contain little or no fines are classified as GW. In these soils, the presence of fines must have no effect on strength characteristics and on free draining characteristics. In addition to the criteria stated previously, this group must have a uniformity coefficient **(C_U)** greater than 4, and the coefficient of curvature (**C_C**) of the soil must be between 1 and 3.

The coefficient of uniformity **C_U**
and coefficient of curvature **C_C**
are expressed as follows:

where

$$C_U = \frac{D_{60}}{D_{10}} \qquad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

D_{10}, ***D_{30}***, *and* ***D_{60}*** *are the grain-size diameters corresponding respectively to 10, 30, and 60% passing on the cumulative grain-size curve.*

SW Group : This group of soils is similar to the GW group except that the predominant grain size is sand rather than gravel. It includes well-graded sands and gravelly sands. The uniformity coefficient of soil in this classification must be greater than 6 and the coefficient of curvature must be between 1 and 3.

GP Group: Soils which classify as gravels and which will not meet the grading requirements of the GW group are placed in the GP group. These soils include poorly graded gravels and sandy gravels having little or no fines.
SP Group: Soils which classify as sands and which will not meet the grading requirements of the SW group are placed in the SP group. These soils include uniformly graded and gap graded sands and gravelly sands.

(b) More Than 12% Pass the 75 µm Sieve

Those coarse-grained soils having more than 12% passing the 75 µm sieve are subdivided by the plasticity characteristics of the fine-grained portion and are given the classification of GM*, GC, SM*, and SC meaning, respectively, Gravel - With Silt Fines, Gravel - With Clay Fines, Sand - With Silt Fines, and Sand - With Clay Fines. The amount of fines in these groups is enough to affect engineering characteristics. Gradation is not a factor in classification.

GM Group: Soils comprising this group are those in which the predominant fine-grained fraction is silt. This group of soils includes silty gravels and mixtures of gravel, sand and silt. Soils which classify as gravels and having a fine-grained portion for which the Atterberg limits (liquid limit and plasticity index) will plot below the A-Line or the plasticity index (I_p) is less than 4 are placed in the GM group. The A-Line is shown and defined on the Plasticity Chart located with the MTC Soil Classification Chart in the Appendix. It is also discussed under Fine-Grained Soils in Section (1.4).

GC Group: Soils which classify as gravels and having a fine-grained portion for which the Atterberg limits will plot above the A-Line and for which the plasticity index is more than 7 are placed in the GC group. This group includes clayey gravels and poorly graded gravel sand- clay mixtures.

SM Group: This group is the same as the GM group except that the predominant coarse grained fraction is sand. The group includes poorly graded sand-silt mixtures and silty sands.

SC Group: This group is the same as the GC group except that the predominant coarse grained fraction is sand. The group includes clayey sands and sands with clays.

(c) Borderline (Between 5% and 12% Pass the 75 µm Sieve)

Those coarse-grained soils containing between 5% and 12% material passing the 75 µm sieve are termed borderline and are given a dual classification such as SW-SM. Also, those coarse grained soils containing more than 12% material passing the 75 µm sieve and for which the Atterberg limits plot in the hatched zone of the Plasticity Chart receive a dual classification such as SM-SC. These double symbols are appropriate to the grading and plasticity characteristics.

(1.4) Fine-Grained Soils

These soils are not subdivided by grain size, but by the properties of plasticity and ompressibility. Fine-grained soils are classified as silt and clay of low, intermediate or high plasticity. Criteria for classification are based upon the relationship between the liquid limit **(w_L)** and the plasticity index (I_p) and are given in the form of a Plasticity Chart. On this chart, for classification, the plasticity index is plotted against the liquid limit.**

*Symbol M for the Swedish word 'Mo', meaning silt.

** Soils of low plasticity generally exhibit low compressibility and those of high plasticity are usually highly compressible.

The A-Line shown on the Plasticity Chart divides inorganic clays from silts and organic soils. Those soils for which the Atterberg limits plot above this line are clays and are designated by the symbol C, while those which plot below it are either silts with the designation M or organic soils with the designation O.

Soils (both silt and clay) which have a liquid limit of less than 35 and low plasticity and compressibility are designated by the symbol L. Soils having a $\mathbf{w_L}$ between 35 and 50 are called silts or clays of intermediate plasticity and compressibility designated by the symbol I. Those soils having a $\mathbf{w_L}$ greater than 50 are termed highly plastic and compressible and are designated by the symbol H. Hence, a soil determined to be a highly plastics and compressible clay is designated as CH, etc.

ML Group: Contains silts having bulky shaped grains, rock flour and sandy silt;

MI Group: Silts of medium compressibility and plasticity are classified in this group, including silty fine sands with some clay, below the A-Line.

MH Group: Silts having flaky shaped grains, including micaceous silts, diatomaceous silts, and elastic silts.

CL Group: Most Kaolinite-type clays are classified in this group, which usually includes gravelly clays, sandy clays, and lean or low plasticity clays.

CI Group: Clays of medium compressibility and plasticity and inorganic silty clays are placed in this category.

CH Group: Montmorillonite-type clays are classified in this group, which includes fat or highly plastic clays, volcanic clays, and bentonites.

Borderline: Those fine-grained soils for which the Atterberg limits plot in the hatched zone on the Plasticity Chart are given the dual classification of CL-ML.

(1.5) Organic Soils

The placement of soils into this group is based upon visual inspection. However, they are subdivided within the group in accordance with their plasticity characteristics. All of these soils should plot below the A-Line on the Plasticity Chart and they are termed organic silts or clays of low (L), intermediate (I), or high plasticity and compressibility (H).

Organic matter tends to decay and thus create more voids in the soil mass. It can be instrumental in chemical alterations which change the physical properties of the soil.

OL Group: This group consists of organic soils having a liquid limit of less than 35. Organic silts and organic silts with sand are usually included in this group.

OI Group: Organic soils having a liquid limit between 35 and 50 belong in this category. Soils in this group are classified as organic clays of medium plasticity or organic silty clays.

OH Group: This group consists of organic soils having a liquid limit of more than 50. Organic clays, and some organic silty clays, will be included in this group.

PT Group : Materials composed mainly of fibrous organic matter are classified separately in this group. Peat and other highly organic soils in various stages of decomposition shall be placed in this category.

Chapter 2

FIELD IDENTIFICATION AND DESCRIPTION OF SOILS

(2.1) General

As implied earlier, soils in nature seldom exist separately as boulders, cobbles, gravel, sand, silt, clay or organic matter but are usually found as mixtures with varying proportions of these components. The MTC Soil Classification System is based on recognition of the type and predominance of the constituents, considering grain size, gradation, plasticity and compressibility. It separates soils into three major divisions - coarse-grained soils, fine-grained soils, and highly organic (peaty) soils and within these divisions it establishes eighteen (18) soil groups.

Two methods for classifying soils have been adopted:

(a) The *visual method* which employs simple manual tests and visual observations to estimate the size and distribution of the coarse-grained soil fractions and to indicate the plasticity characteristics of the fine-grained fractions. The visual method is used predominantly for field classification purposes.

(b) The *laboratory method* which, in addition to visual observations, employs laboratory determinations of the particle-size distribution and plasticity of the fines to assist in assigning soil group names. The laboratory method is used only when confirmation or precise description is required. It is also useful as a training aid for beginners. (See Chapter 3)

(2.2) Visual Method

No special apparatus or equipment is required for the visual identification and/or description of soils. However, the following items will facilitate the procedure:

(a) A rubber syringe or a small oil can.
(b) A supply of clean water.
(c) Small bottle of dilute hydrochloric acid (HCI).
(d) Classification chart (Appendix).

The classification of a soil by this method is based on visual observations and estimates of its engineering behaviour in a remoulded state. If the soil is to be utilized in a foundation, the condition of the soil in the undisturbed state must also be described, as discussed in section 2.7. The procedure for visual classification of a soil is, in effect, a process of simple elimination, beginning on the left side of the classification chart and working to the right until the proper group name is obtained. It is emphasized that the group name must be supplemented by a detailed description which points out peculiarities of a particular soil and differentiates it from others in the same group.

Many natural soils will have properties not clearly associated with any one soil group, but which are common to two or more groups. Or they may be near the borderline between two groups, either in percentages of the various sizes or in plasticity characteristics. For this substantial number of soils, boundary classifications are assigned and boundary group symbols are used. The two group symbols most nearly indicating the proper soil description are recorded. These are connected by a hyphen as, for example, GW-GC, SC-CL, ML-CL, and others.

Preparation of Sample
The following procedure should be used:

(a) Select a representative sample of the soil.
(b) Estimate the maximum particle size in the sample.
(c) Estimate the percentage by mass of particles larger than 75 mm.
(d) If more than half of the particles are larger than 75 mm, classify the soil as boulders and/or cobbles as explained under section (1.1).
(e) If less than half of the particles are larger than 75 mm, remove all these particles from the sample. Only that fraction of the sample smaller than 75 mm is classified.

Division between Coarse and Fine-Grained Particles

Classify the sample as coarse-grained or fine-grained by estimating the percent by mass, of individual particles which can be seen by the naked eye. Soils containing more than 50% visible particles are coarse-grained soils. Soils containing less than 50% visible particles are fine-grained soils.

(2.3) Visual Procedure for Coarse-Grained Soils

If it has been determined that the soil is predominantly coarse-grained, it is further identified by estimating the percentages of grains in the size range (a) 75 mm to 4.75 mm, and (b) 4.75 mm to 75 µm. If percentage (a) is greater than (b), the soil will finally be placed in one of the GRAVEL groups (Symbol G), and if percentage (b) is greater than (a), it will be placed in one of the SAND groups (Symbol S), as described below.

Gravel **-** Gravels are further identified as being CLEAN (containing little or no fines) or DIRTY (containing appreciable fines) by estimating the percentage of grains not visible to the naked eye. If the soil is obviously clean and pervious, the final classification will be either:

WELL-GRADED GRAVEL (GW) if there is a good representation of all particle sizes, or POORLY-GRADED GRAVEL (GP) if there is an absence of intermediate particle sizes or an excess of one size.

If the soil is obviously dirty and relatively impervious, the final classification will be either SILTY GRAVEL (GM) if the fines have no or low plasticity, or CLAYEY GRAVEL (GC) if the fines are of low to medium plasticity.

Sand **-** If it was determined that the soil would finally be placed in one of the SAND groups, the same procedure would be applied as for gravels except that the word SAND replaces GRAVEL in the groups and the Symbol S replaces G. Thus, the obviously clean sands will be classified as either:

WELL-GRADED SAND (SW) or

POORLY-GRADED SAND (SP)

and the obviously dirty sands will be classified as SILTY SAND (SM) if the fines have no or low plasticity, or

CLAYEY SAND (SC) if the fines are of low to medium plasticity.

Coarse-Grained Soils - Descriptive information for coarse-grained soils may include:

(a) Typical name.
(b) Maximum size of particles, percentage of cobbles and boulders which are not included in the classification.
(c) Approximate percentage of gravel, sand and fines.
(d) Description of average size of sand or gravel; whether coarse, medium .or fine according to the groups as follows:

Coarse gravel - 75 mm to 26.5 mm
Fine gravel - 26.5 mm to 4.75 mm
Coarse sand - 4.75 mm to 2.00 mm
Medium sand - 2.00 mm to 425 µm
Fine Sand - 425 µm to 75 µm

(e) Description of the shape of the grains; rounded, subrounded, angular, subangular. (See Figure 2)
(f) The surface coatings, cementation, and hardness of the grains and possible breakdown when compacted.
(g) The colour and organic content.
(h) Plasticity of fines.
(i) Geologic origin or formation, if known.
(j) Group symbol.

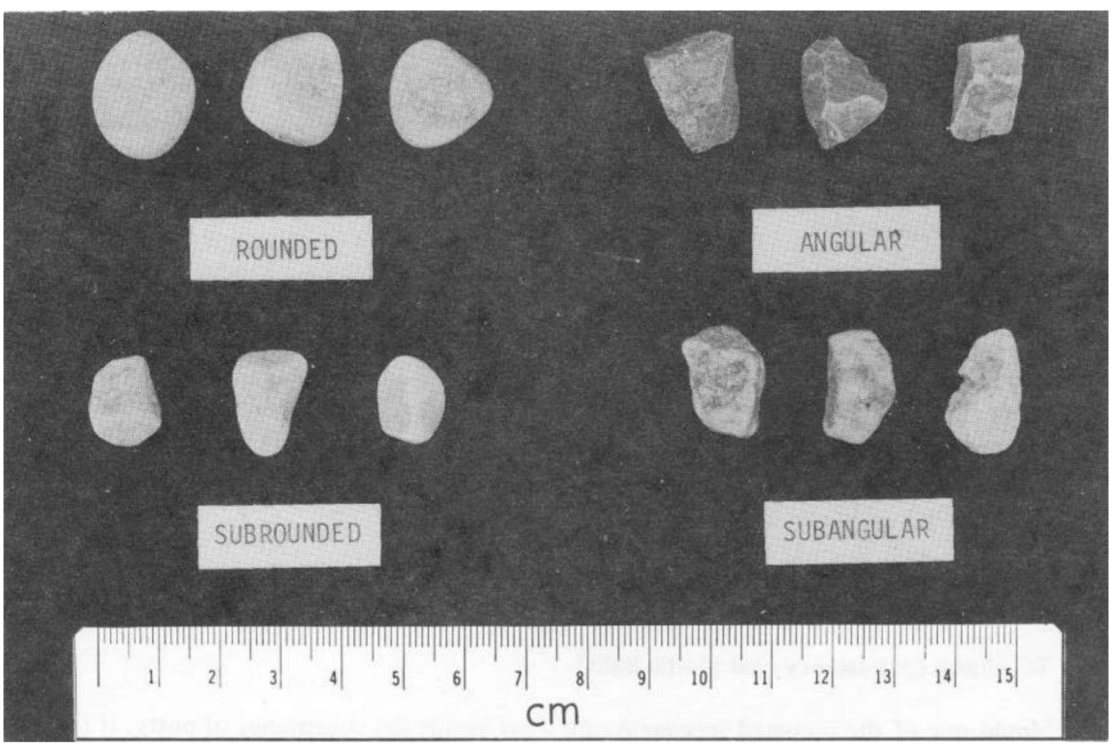


Figure 2. Typical Shapes of Gravels

(2.4) Visual Procedure for Fine-Grained Soils

If it has been determined that the soil is predominantly fine-grained (smaller than 75 µm), it is further identified according to its plasticity characteristics. The manual tests for dry strength, dilatancy, and toughness are performed as described below, and the results of these tests will determine which of the nine (9) standard group terminologies will apply. The same procedures are used to identify the fine-grained fraction of coarse-grained soils to determine whether they are silty or clayey.

(2.4.1) Manual Tests

The tests for identifying fine-grained soils in the laboratory are performed on that fraction of the soil finer than 425 µm. Therefore, the first step in classifying these soils in the field is to select a representative sample and remove by hand as much of the large particle size material as is practical. Then prepare three small specimens by moistening until they can easily be rolled into a ball. Perform the tests listed below, carefully noting the behaviour of each soil pat during each test. Operators with considerable experience find that it is not necessary in all cases to prepare all three pats. For example, if the soil contains dry lumps, the dry strength can be readily determined without preparing a pat for this particular purpose.

(a) Dry Strength (crushing resistance)

Air dry one of the prepared specimens. Then determine its resistance to crumbling and powdering between the fingers. This resistance, called "dry strength", is a measure of the plasticity of the soil and is influenced largely by the colloidal fraction contained. The dry strength is designated as "slight" if the dried pat can be easily powdered, "medium" if considerable finger pressure is required, and "high" if it cannot be powdered at all.

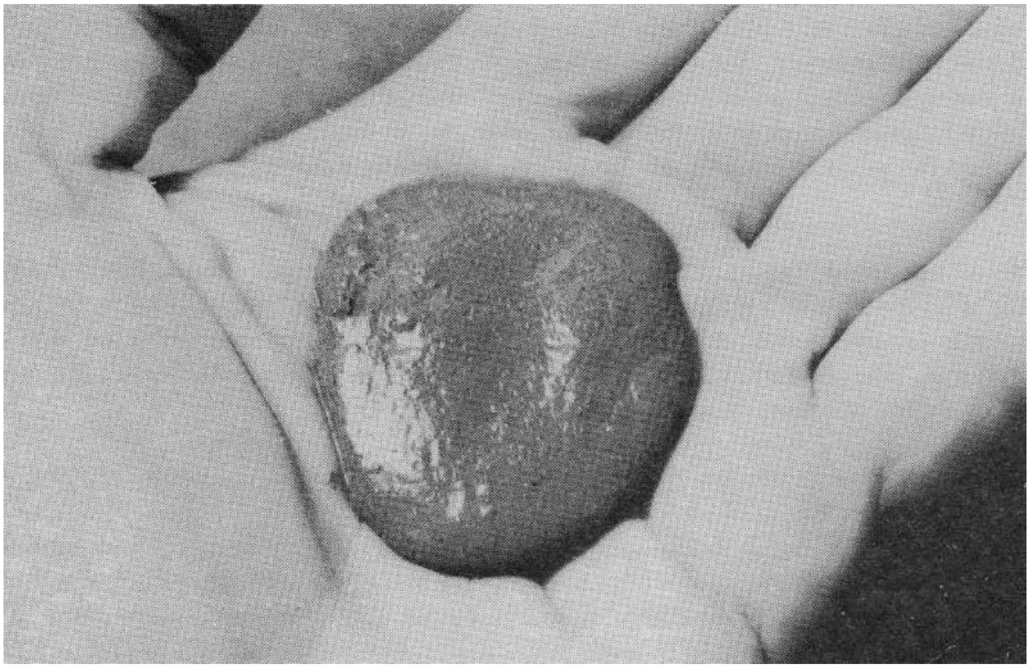
High dry strength is characteristic of clays. Silts possess 'none to slight' dry strength.

Note: The presence of high-strength water soluble cementing materials, such as calcium carbonates' or iron oxides, may cause high dry strengths. Non-plastic soils, such as caliche, coral, crushed limestone, or soils containing carbonaceous cementing agents, may have high dry strengths, but this can be detected by the effervescence caused by the application of dilute hydrochloric acid (see acid test below).

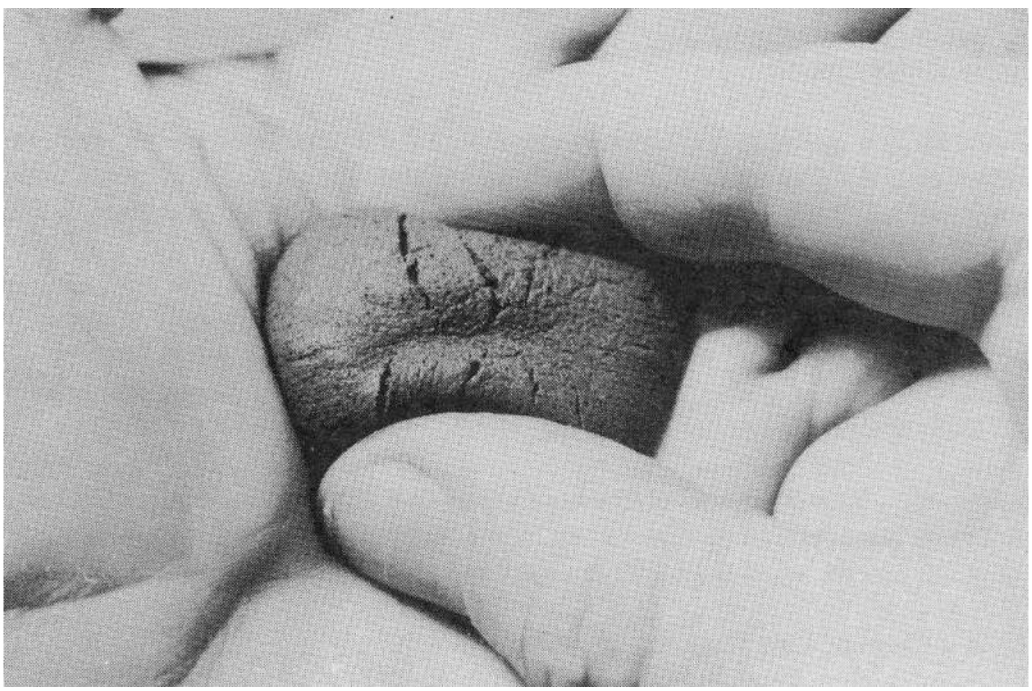
(b) Dilatancy (reaction to shaking)

Add enough water to nearly saturate one of the soil pats. Place the pat in the open palm of one hand and shake horizontally striking vigorously against the other hand several times. Squeeze the pat between the fingers. The appearance and disappearance of the water with shaking and squeezing is referred to as "reaction" (Figure 3**).** This reaction is called "quick" if water appears and disappears rapidly, "slow" if water appears and disappears slowly, and "no reaction" if the water condition does not appear to change.

Fine clean sands give the quickest reaction, while plastic clays have no reaction. Silts usually show 'slow to quick "reaction.



(a) Reaction to Shaking



(b) Reaction to Squeezing

Figure 3. Test for Dilatancy

(c) Toughness (consistency near plastic limit)

Mould one of the prepared specimens into a pat having the consistency of putty. If too dry, water must be added and if too sticky the specimen should be allowed to partially dry. Roll the pat on a smooth surface or between the palms into a thread about 3 mm in diameter. Fold and reroll the thread repeatedly to 3 mm diameter so that its moisture content is gradually reduced until the 3 mm thread just crumbles (Figure 4). The moisture content at this time is called "the plastic limit", and the resistance to moulding at the plastic limit is called the "toughness". After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles. If the lump can still be remoulded slightly drier than the plastic limit and if high finger pressure is required to roll the thread between the palms of the hands, the soil is described as possessing "high toughness".

"Medium toughness" is indicated by a medium tough thread, and a lump formed of the threads slightly below the plastic limit will crumble; while "slight toughness" is indicated by a weak thread that breaks easily and cannot be lumped together when drier than the plastic limit. The thread will be soft and spongy. Nonplastic soils cannot be rolled into a thread of 3 mm diameter at any moisture content.

(d) Organic Content and Colour

Fresh, wet organic soils usually have a distinctive odour of decomposed organic matter. This odour can be made more noticeable by heating the wet sample. Another indication of the organic material is the distinctive dark colour. Dry, inorganic clays develop an earthy odour upon moistening, which is distinctive from that of decomposed organic matter.

(e) Additional Identification Tests

Acid Test - This test, using dilute hydrochloric acid (HCl), is primarily a test for the presence of calcium carbonate. Two or three drops of the acid solution on soil or rock will cause effervescence if the material contains calcium carbonate. For soils with high dry strength, a strong reaction indicates that the strength may be due to calcium carbonate as a cementing agent, rather than colloidal clay. The results of this test should be included in the soil description, if pertinent.

Shine - This is a quick supplementary procedure for determining the presence of clay. The test is performed by rubbing a lump of dry or slightly moist soil with the thumb. A shiny surface imparted to the soil indicates highly plastic clay, while a dull surface indicates silt or clay of low plasticity.

Grit - Silts feel gritty to the teeth, clays do not.

Stickiness - Clays are sticky. Silts are not. Silts can be readily dusted from the hands after drying. Clays cannot be readily dusted off.

Miscellaneous - Other criteria undoubtedly can be developed by the individual as he gains experience in classifying soils. For example, differentiation between some of the fine-grained soils depends largely upon the experience in the "feel" of the soils.

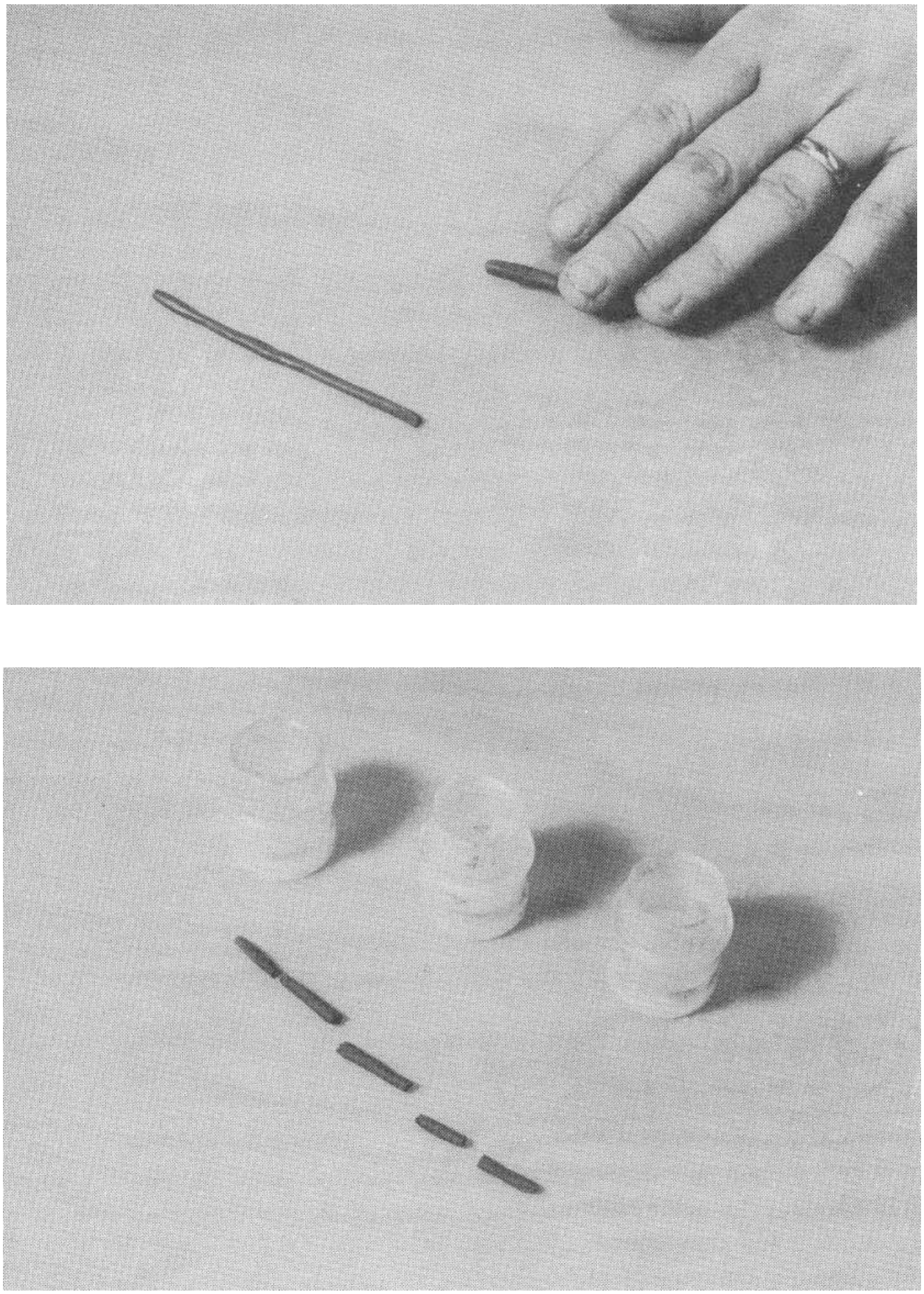


Figure 4. Test for Plastic Limits

(2.4.2) Silts and Clays of Low Plasticity (Symbol L**)**

The following three groups in this category are commonly thought of as soils possessing low plasticity. Various combinations of results of the manual identification tests indicate which grouping is proper for the soil in question.

(a) INORGANIC SILT (ML) normally possesses no dry strength, quick dilatancy, and no toughness.
(b) INORGANIC CLAY (CL) usually has medium to high dry strength, none to very slow dilatancy, and medium toughness.
(c) ORGANIC SILT OR CLAY (OL) generally has slight to medium dry strength, slow dilatancy and slight toughness. Organic matter must be present in sufficient amounts to influence the soil properties in order for a soil to be placed in this group.

(2.4.3) Silts and Clays of Intermediate Plasticity (Symbol I)

(a) INORGANIC SILT (MI) has "none to slight" dry strength, slow to quick dilatancy, and slight toughness.
(b) INORGANIC CLAY (CI) usually has high dry strength, no dilatancy, and medium to high toughness.
(c) ORGANIC SILT OR CLAY (OI) generally has slight to medium dry strength, very slow dilatancy and slight toughness. Organic matter must be present in sufficient amounts to influence soil properties in order for a soil to be placed in this group.

(2.4.4) Elastic Silts and Fat Clays (Symbol H)

(a) INORGANIC SILT (MH) is generally very absorptive, has slight to medium dry strength, "none to slow" dilatancy, and medium toughness. Some inorganic soils (such as kaoline which may be a clay from a mineralogical standpoint) possessing medium dry strength and toughness will fall in this group.
(b) INORGANIC CLAY (CH) always possesses high to very high dry strength, no dilatancy, and usually high toughness.
(c) ORGANIC SILT OR CLAY (OH) normally has medium to high dry strength, none to very slow dilatancy, and slight to medium toughness. Organic matter must be present in sufficient amounts to influence soil properties in order for a soil to be placed in this group.

2.5 Visual Procedure for Highly Organic Soils (Symbol P_t)

Highly organic material is readily identified by colour, odour, spongy feel and it may be fibrous or amorphous. These materials are not broken down further.

Partly organic soils tend to have the properties of their inorganic component but will exhibit high compressibility in relation to their organic content. These soils are further identified based on their inorganic components using the procedures described above.

Fine-Grained and Organic Soils **-** Descriptive information for remoulded fine-grained soils may include:

(a) Typical name.
(b) Amount and maximum size of coarse grains.
(c) Colour and organic content.
(d) Natural moisture content, drainage conditions.
(e) Plasticity characteristics.
(f) Geologic origin or local name, if known, (e.g. Glacial Till, Leda Clay).
(g) Group Symbol.

2.6 Guide for Field Description Procedure

In the field the soil may be described using the following steps. Abbreviations (see Table 1 in the Appendix) are used for pavement structure investigations owing to the numerous borehole logs required for this type of work.

1st Step: Decide whether the sample is predominantly coarse-grained, fine-grained or organic. In borderline cases dual descriptions may be used.

2nd Step: Identify the principal component based on grain size if coarse-grained, or oil behaviour if fine-grained, and use as a noun in the soil description.

Example: *Medium sand (Med Sa)*
Clay (Cl)

3rd Step: Identify the second component. Does it have a substantial effect on the total sample (in the range of 10% - 15% of the total mass of a coarse-grained material).

If yes, use it as an adjective preceding the noun in the soil description.

Example: *Silty fine sand (Si F Sa)*
Silty fine sand to fine sandy silt (Si F *SA-Sa* F *Si)*
Silty clay (Si Cl)

If no, treat as in step 4.

4th Step: Are there minor components which still have a significant effect on the sample?

If yes, identify by using the word "with-".

Example: *Silty fine sand with gravel (Si F Sa with Gr)*
Sand with silt (Sa with Si)

5th Step: Are there components which have only a minor effect on the sample?

If yes, identify by using the word "trace-".

Example: *Silty fine sand trace of clay (Si F Sa Tr Cl)*
Silty fine to medium sand with gravel trace of clay (Si F-Med *Sa with Gr Tr Cl)*
Coarse sand trace of silt (Co Sa Tr Si)

6th Step: Describe the colour of the soil.

Example: *Grey silty sand (Gry Si Sa)*

7th Step: Describe the moisture content. The field moisture content may be defined as; dry, moist or wet.

Dry: a dry soil has a moisture content well below optimum ($w < w_{opt}$)

Moist: a soil with a moisture content at or near optimum ($w \approx w_{opt}$)

Wet: a soil with a moisture content well above optimum ($w > w_{opt}$) .

Saturated: denotes the moisture condition of a soil below the water table. It may be below, at or above the optimum moisture content.

Example: *Grey silty sand, moist (Gry Si Sa, moist)*

2.7 Description of Foundation Soils

The in-place condition of soils which are to be utilized as foundations for structures assumes primary importance in soil classification. Borehole logs of foundation explorations and descriptions of undisturbed samples, therefore, must emphasize such in-situ conditions of the soil. It is necessary for the classifier to present a complete word picture describing the soil as it exists in-situ, in addition to placing it in the proper group. The following properties should be described for foundation soils in addition to those properties listed earlier.

(1) Permeability or drainage properties in the natural condition.
(2) Structure (stratified, flocculent, honeycomb, etc.).
(3) Type and degree of cementation.
(4) Degree of compactness or denseness.

For fine-grained soils the consistency in undisturbed and remoulded states should also be described.

The consistency of a cohesive, and the denseness of a non-cohesive soil in the field, are usually determined by Field Vane Tests or Standard Penetration Tests respectively.

Consistency - Is described on the basis of undrained shear strength (c_U)* as follows:

Consistency	**c_u (kPa)**
Very soft	0 - 12
Soft	12 -25
Firm	25 - 50
Stiff	50 - 100
Very stiff	100 - 200
Hard	> 200

Denseness - is described as indicated by Standard Penetration Test 'N' values as follows:

Denseness	**'N' (Blows / 0.3 m)**
Very loose	0 - 5
Loose	0 - 10
Compact	10 - 30
Dense	30 - 50
Very dense	> 50

The principal engineering properties of the various soil groups are tabulated in the Appendix.

*c_u is the symbol for apparent cohesion, which, in an undrained condition with saturated cohesive soils, is also called "undrained shear strength". Shear strength at failure $\tau_f = c_u + \sigma \tan \sigma_u$; but in the undrained case $\sigma_u = 0$, thus $\tau_f = c_u$

Chapter 3

LABORATORY CLASSIFICATION OF SOILS

Laboratory classification will, in general, be applied to those materials taken from natural deposits. All processed materials, and some materials from natural deposits will be tested and classified under the provisions of MTC Specification Division 10 for aggregates (e.g., HL1 and 3, Granular 'B', etc.).

Precise delineation of the soil groups may be obtained in the laboratory by means of grain-size analysis and Atterberg limits. For foundation or slope stability investigations, these tests are carried out as a matter of course together with other extensive testing, such as the measurement of strength and compressibility.

Laboratory tests may also be used as training for field personnel to improve their ability in estimating grain size percentage and degree of plasticity.

3.1 Test Apparatus

Descriptions of the 'apparatus required may be found in the following test methods described in the MTC Laboratory Testing Manual:

LS-700 Method of Dry Preparation of Soil ...
LS-600 Method of Preparation of Aggregates ...
LS-703 Method of Test for Liquid Limit of Soils ...
LS-704 Method of Test for Plastic Limit ...
LS-602 Method of Test for Sieve Analysis ...
LS-702 Method of Test for Particle Size Analysis ...

3.2 Test Samples

Test samples shall be prepared as follows:

(a) The sample as received .from the field shall be thoroughly dried in air or a drying oven not exceeding 60°C if it is predominantly fine-grained. Predominantly coarse grained material may be dried in an oven at 110 ± 5°C.

(b) The dried sample is separated into fractions on a 4.75 mm sieve if it is predominantly sand, silt or clay and on a 26.5 mm sieve if it has a substantial portion larger than 26.5 mm. The coarse portions are sieved in each case and the masses retained on each sieve recorded.

(c) Samples passing the 4.75 mm sieve are split and one portion is ground in a mortar with a rubber pestle and passed through a 2.00 mm sieve. The mass of material passing the 2.00 mm sieve may be split further to produce one sample for particle size analysis and one sample to be ground further to produce sufficient material passing the 425µm sieve for Atterberg limit or organic tests.

(d) Samples passing the 26.5 mm sieve are placed in the Gilson sieving apparatus to obtain the grain-size distribution for particles from 19.00 mm to 4.75 mm. The material passing the

4.75 mm sieve is split to obtain a representative sample for the sieve analysis of the medium to fine sand.

(e) When all the tests have been completed, .the grain-size distribution curve is plotted and the classification procedure may be carried out.

3.3 Procedure for Classification of Coarse-Grained Soils

(a) Classify the material as gravel (G) if 50% or more of the fraction retained on the 75 µm sieve is larger than 4.75 mm.

(b) Classify the material as sand (S) if more than 50% of the fraction retained on the 75 µm sieve is smaller than 4.75 mm.

(c) If less than 5% of the sample passes the 75 µm sieve, compute the coefficient of uniformity (CU) and the coefficient of curvature (CC) as explained under Section (1.3).

(d) Classify the sample as well-graded gravel (GW) or well-graded sand (SW), if C_U is greater than 4 for gravel and greater than 6 for sand, and C_C is between 1 and 3. Classify the sample as poorly-graded gravel (GP) or poorly-graded sand (SP) if either the C_U or C_C criteria are not satisfied.

(e) If more than 12% of the sample passes the 75 µm sieve, the liquid limit and plasticity index of the pass 425 µm portion should be determined.

(f) Classify the sample as silty gravel (GM) or silty sand (SM) if the results of the limit tests show the fines to be silty i.e., below the A-line or with plasticity index (I_p) less than 4. Classify the sample as clayey gravel (GC) or clayey sand (SC) if the results of the limit tests plot above the A-line and the plasticity index (I_p) is greater than 7.

(g) If the fines are intermediate between silt and clay, i.e., plot on or close to the A-line and with plasticity index (I_p) in the range of 4-7 (the hatched area in the plasticity chart), the sample should be given a borderline classification, such as GM-GC or SM-SC.

(h) If 5 to 12% of the sample passes the 75 µm sieve, the material should be given a borderline classification based on gradation and plasticity characteristics, such as GW-GC or SP-SM, In doubtful cases, the rule is to favour the less plastic classification.

Example: a gravel with 10% fines. a C_U of 20, a C_C of 2 and a plasticity index of 6 would be classified as GW-GM rather than GS-GC.

3.4 Procedure for Classification of Fine-Grained Soils

(a) If results of Atterberg limit tests plot above the A-line and the plasticity index is greater than 7, classify the sample as clay of high plasticity (CH) if the liquid limit is above 50; intermediate plasticity (CI) if the liquid limit is between 35 and 50 and low plasticity (CL) if the liquid limit is below.

(b) If results of Atterberg limit tests plot below the A-line or the plasticity index is less than 4, classify the sample as silt of high plasticity (MH) if the liquid limit is above 50;

intermediate plasticity (MI) if the liquid limit is between 35 and 50 and low plasticity (ML) if the liquid limit is below 35.

(c) If the sample has a dark colour and an organic odour when moist and warm, the results of Atterberg limit tests plot below the A-line or the plasticity index is less than 4, classify the sample as organic clay or silty-clay of high plasticity (OH) if the liquid limit is above 50 and intermediate plasticity (OI) if the liquid limit is between 35 and 50. If the liquid limit is less than 35, classify the material as organic silt or organic silt with sand (OL).

(d) If the organic content is in doubt, a determination should be made by oxidation or reduction. If this cannot be carried out, Atterberg limit tests should be performed on natural, air-dried or oven-dried soil. Marked differences in the liquid limit or plasticity index would indicate a high organic content.

(e) If the results of Atterberg limit tests fall close to or on the A-line or in the hatched area of the plasticity chart **(**$\mathbf{w_L}$ < 30%; I_p = 4-7%) the sample should be classified as borderline such as CL-ML or CI-MI.

(f) If the plot of the liquid limit versus plasticity index falls close to or on the vertical line of 35 or 50% liquid limit, the sample should be given a borderline classification such as CICH, MI-MH, CL-CI or ML-MI. In doubtful cases, the rule is to favour the more plastic classification.

Example: *a fine-grained soil with a liquid limit* of *50 and a plasticity index* of *22 would be classified as CH-MH rather than CI-MI.*

REFERENCES

1. AASHTO. Washington, D.C. (1978), *Manual on Foundation Investigations* (pp B-26 to B-36).

2. ASTM. (1978), *Annual Book of ASTM Standards. Part 19.* D 653 (Reapproved 1973); D-2487 (Reapproved 1975); D 2488 (Reapproved 1975).

3. Ministry of Transportation and Communications, Ontario (1 977), *Geotechnical Metric Guide* (Revised edition).

4. Ministry of Transportation and Communications, Ontario (1979), *Laboratory Testing Manual.*

5. National Research Council of Canada (1955), *Guide to the Field Description of Soils for Engineering Purposes.*

6. Road Research Laboratory. Great Britain (1954), *Soil Mechanics for Road Engineers.* Chapter 4.

7. Soil Mechanics Bureau. N.Y. State DOT. (1978), *Soil Description Procedure.*

8. U.S. Department of the Interior. Bureau of Reclamation. Denver, Colorado, (1 963), *Unified Soil Classification System.*

(Blank Page)

APPENDIX

ABBREVIATIONS FOR BORING AND TEST DATA

Amor	— Amorphous
Asph	— Asphalt
BR	— Bedrock
Blk	— Black
Bl	— Blue
Bld(y)	— Boulder (y)
Blds	— Boulders
BU	— Break Up
Br	— Brown
Cl	— Clay
Co	— Coarse
Cob	— Cobbles
Comp	— Compact
Conc	— Concrete
Contam	— Contaminated
Cord	— Corduroy
Cr	— Crushed
Dk	— Dark
Decomp	— Decomposed
E	— Earth
Fib	— Fibrous
w	— Field Moisture Content
F	— Fine
Fr Wat	— Free Water
FB	— Frost Boil
FH	— Frost Heave
Gran	— Granular
Gr	— Gravel(ly)
Grn	— Green
Grey	— Grey
Hi	— Highly
HM	— Hot Mix
Lt	— Light
Liq	— Liquid
w_L	— Liquid Limit
Matl	— Material
Max	— Maximum
MDD	— Maximum Dry Density
MWD	— Maximum Wet Density
Med	— Medium
Mod	— Moderate
Mott	— Mottled
Mul	— Mulch
NFP	— No Further Progress
NFP (Blds)	— No Further Progress (Boulders)
Num	— Numerous
Occ	— Occasional
w_{opt}	— Optimum Moisture Content
Ora	— Orange
Org	— Organic
Org M	— Organic Mattter
Ob	— Overburden
Pavt	— Pavement
Pedo	— Pedological
Pen Mac	— Penetration Macadam
w_P	— Plastic Limit
I_P	— Plasticity Index
PSty	— Polystyrene
Poss	— Possible
PST	— Prime & Surface Treated
Quant	— Quantity
Reinf	— Reinforced
RF	— Rock Fill
Sa	— Sand
Sat	— Saturated
Sh Rk	— Shot Rock
Si	— Silt(y)
Sl(y)	— Slight(ly)
D_R	— Relative Density
Stks	— Streaks
Surf	— Surface
Temp	— Temperature
Tps	— Topsoil
Tr	— Trace
Unreinf	— Unreinforced
Varv	— Varved
VF	— Very Fine
WT	— Water Table
Weath	— Weathered
Wd(y)	— Wood(y)
Yel	— Yellow

SUSCEPTIBILITY TO FROST HEAVING

HSFH — High
BSFH — Borderline
LSFH — Low

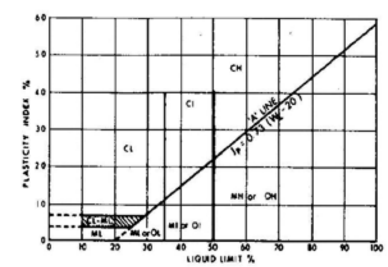
M.T.C. SOIL CLASSIFICATION

FIELD IDENTIFICATION PROCEDURES (EXCLUDING PARTICLES LARGER THAN 75 mm AND BASING FRACTIONS ON ESTIMATED MASS)			GRP. SYMB.	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA		
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN 75 µm) — GRAVELS (MORE THAN HALF OF COARSE FRACTION IS LARGER THAN 4.75 mm)	CLEAN GRAVELS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZE & SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZE	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES; LITTLE OR NO FINES	GIVE TYPE; NAME, IF NECESSARY; INDICATE APPROX. % OF SAND & GRAVEL; MAX. SIZE; ANGULARITY, SURFACE CONDITION, & HARDNESS OF THE COARSE GRAINS; LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION; & SYMBOL IN PARENTHESIS.	DETERMINE PERCENTAGES OF GRAVEL & SAND FROM GRAIN SIZE CURVE. DEPENDING ON PERCENTAGE OF FINES (FRACTION SMALLER THAN 75 µm) COARSE GRAINED SOILS ARE CLASSIFIED AS FOLLOWS: LESS THAN 5% GW, GP, SW, SP; MORE THAN 12% GM, GC, SM, SC; 5% TO 12% BORDERLINE CASES REQ. USE OF DUAL SYMBOLS	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN ONE AND 3	
		PREDOMINANTLY ONE SIZE OF A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING	GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES; LITTLE OR NO FINES			NOT MEETING ALL GRADATION REQUIREMENTS FOR GW	
	GRAVEL WITH FINES (APPRECIABLE AMOUNT OF FINES)	NON-PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES			ATTERBERG LIMITS BELOW A-LINE, OR I_p LESS THAN 4	ABOVE A-LINE WITH I_p BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
		PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES			ATTERBERG LIMITS ABOVE A-LINE WITH I_p GREATER THAN 7	
SANDS (MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN 4.75 mm)	CLEAN SANDS (LITTLE OR NO FINES)	WIDE RANGE IN GRAIN SIZES & SUBSTANTIAL AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES	SW	WELL GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES	FOR UNDISTURBED SOILS ADD INFORMATION ON STRATIFICATION, DEGREE OF COMPACTNESS, CEMENTATION, MOISTURE CONDITIONS & DRAINAGE CHARACTERISTICS.		$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN ONE AND 3	
		PREDOMINANTLY ONE SIZE OR A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING	SP	POORLY GRADED SANDS, GRAVELLY SANDS; LITTLE OR NO FINES			NOT MEETING ALL GRADATION REQUIREMENTS FOR SW	
	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	NON-PLASTIC FINES (FOR IDENTIFICATION PROCEDURES SEE ML BELOW)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES			ATTERBERG LIMITS BELOW A-LINE OR I_p LESS THAN 4	ABOVE A-LINE WITH I_p BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
		PLASTIC-FINES (FOR IDENTIFICATION PROCEDURES SEE CL BELOW)	SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES			ATTERBERG LIMITS ABOVE A-LINE WITH I_p GREATER THAN 7	

USE GRAIN SIZE CURVE IN IDENTIFYING THE FRACTIONS AS GIVEN UNDER FIELD IDENTIFICATION

IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN 425 µm

FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN 75 µm) (75 µm IS ABOUT THE SMALLEST PARTICLE VISIBLE TO THE NAKED EYE)		DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)	GRP. SYMB.	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS
SILTS AND CLAYS	LIQUID LIMIT LESS THAN 35%	NONE	QUICK	NONE	ML	INORGANIC SILTS & SANDY SILTS OF SLIGHT PLASTICITY, ROCK FLOUR	GIVE TYPE; NAME, IF NECESSARY, INDICATE DEGREE & CHARACTER OF PLASTICITY, AMOUNT & MAXIMUM SIZE OF COARSE GRAINS, COLOUR IN WET CONDITION, ODOUR, IF ANY, LOCAL OR GEOLOGIC NAME & OTHER PERTINENT DESCRIPTIVE INFORMATION & SYMBOL IN PARENTHESIS.
		MEDIUM TO HIGH	NONE TO VERY SLOW	MEDIUM	CL	SILTY CLAYS (INORGANIC), GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	
		SLIGHT TO MEDIUM	SLOW	SLIGHT	OL	ORGANIC SILT OF LOW PLASTICITY, ORGANIC SANDY SILTS	
	LIQUID LIMIT BETWEEN 35% AND 50%	NONE TO SLIGHT	SLOW TO QUICK	SLIGHT	MI	INORGANIC COMPRESSIBLE FINE SANDY SILT WITH CLAY OF MEDIUM PLASTICITY, CLAYEY SILTS	FOR UNDISTURBED SOILS AND INFORMATION ON STRUCTURE, STRATIFICATION, CONSISTENCY IN UNDISTURBED & REMOULDED STATES, MOISTURE & DRAINAGE CONDITIONS.
		HIGH	NONE	MEDIUM TO HIGH	CI	SILTY CLAYS (INORGANIC) OF MEDIUM PLASTICITY	
		SLIGHT TO MEDIUM	VERY SLOW	SLIGHT	OI	ORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	
	LIQUID LIMIT GREATER THAN 50%	SLIGHT TO MEDIUM	SLOW TO NONE	MEDIUM	MH	INORGANIC SILTS, HIGHLY COMPRESSIBLE MICACEOUS OR DIATOMACEOUS FINE SANDY SILTS, ELASTIC SILTS	
		HIGH TO VERY HIGH	NONE	HIGH	CH	CLAYS (INORGANIC) OF HIGH PLASTICITY, FAT CLAYS	
		MEDIUM TO HIGH	NONE TO VERY SLOW	SLIGHT TO MEDIUM	OH	ORGANIC CLAYS OF HIGH PLASTICITY	
HIGHLY ORGANIC SOILS		READILY IDENTIFIED BY COLOUR, ODOUR, SPONGY FEEL & FREQUENTLY BY FIBROUS TEXTURE			Pt	PEAT & OTHER HIGHLY ORGANIC SOILS	



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

BOUNDARY CLASSIFICATIONS: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS. FOR EXAMPLE GW-GC, WELL GRADED GRAVEL-SAND MIXTURE WITH CLAY BINDER

TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	PERMEABILITY WHEN COMPACTED	STRENGTH WHEN COMPACTED	COMPRESSIBILITY WHEN COMPACTED	WORKABILITY AS A CONSTRUCTION MATERIAL	SCOUR RESISTANCE	SUSCEPTIBILITY TO SURFICIAL EROSION	SUSCEPTIBILITY TO FROST ACTION	DRAINAGE CHARACTERISTICS
WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	GP	VERY PERVIOUS	GOOD	NEGLIGIBLE	GOOD	MEDIUM	NEGLIGIBLE	NEGLIGIBLE	EXCELLENT
SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES	GM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	NEGLIGIBLE	GOOD	LOW TO MEDIUM	SLIGHT	SLIGHT	FAIR TO SEMI IMPERVIOUS
CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	GC	IMPERVIOUS	GOOD TO FAIR	VERY LOW	GOOD	MEDIUM	SLIGHT	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SW	PERVIOUS	EXCELLENT	NEGLIGIBLE	EXCELLENT	LOW TO MEDIUM	SLIGHT	NEGLIGIBLE	EXCELLENT
POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	SP	PERVIOUS	GOOD	VERY LOW	FAIR TO GOOD	LOW TO MEDIUM	MODERATE	NEGLIGIBLE TO SLIGHT	EXCELLENT
SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	SM	SEMI-PERVIOUS TO IMPERVIOUS	GOOD	LOW	FAIR	LOW	MODERATE	SLIGHT TO MODERATE	FAIR TO SEMI IMPERVIOUS IMPERVIOUS
CLAYEY SANDS, POORLY GRADED SAND WITH SOME CLAY MIXTURES	SC	IMPERVIOUS	GOOD TO FAIR	LOW	GOOD	VERY LOW TO LOW	MODERATE TO SLIGHT	NEGLIGIBLE	PRACTICALLY IMPERVIOUS
INORGANIC SILTS AND SANDY SILTS OF SLIGHT PLASTICITY, ROCK FLOUR	ML	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM	FAIR	VERY LOW	SEVERE	SEVERE	FAIR TO POOR
INORGANIC CLAYEY SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS	CL	IMPERVIOUS	FAIR	MEDIUM	GOOD TO FAIR	LOW TO MEDIUM	SLIGHT TO MODERATE	MODERATE TO SEVERE	PRACTICALLY IMPERVIOUS
ORGANIC SILTS OF LOW PLASTICITY	OL	SEMI-PERVIOUS TO IMPERVIOUS	POOR	MEDIUM	FAIR TO POOR	VERY LOW TO LOW	SEVERE	SEVERE	POOR
INORGANIC COMPRESSIBLE SILTS OF MEDIUM PLASTICITY	MI	SEMI-PERVIOUS TO IMPERVIOUS	FAIR	MEDIUM TO HIGH	FAIR TO POOR	LOW	MODERATE	MODERATE TO SEVERE	FAIR TO POOR
INORGANIC SILTY CLAYS OF MEDIUM PLASTICITY	CI	IMPERVIOUS	FAIR TO POOR	HIGH	FAIR	LOW TO MEDIUM	SLIGHT	MODERATE TO SEVERE	SEMI IMPERVIOUS TO PRACTICALLY IMPERVIOUS
ORGANIC SILTY CLAY OF MEDIUM PLASTICITY	OI	SEMI-PERVIOUS TO IMPERVIOUS	POOR	HIGH	POOR	VERY LOW TO LOW	SEVERE	MODERATE TO SEVERE	POOR TO PRACTICALLY IMPERVIOUS
INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	MH	SEMI-PERVIOUS TO IMPERVIOUS	FAIR TO POOR	HIGH	POOR	VERY LOW	MEDIUM	SEVERE	POOR
INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	CH	IMPERVIOUS	POOR	HIGH	FAIR TO POOR	LOW TO MEDIUM	SLIGHT TO NEGLIGIBLE	NEGLIGIBLE	PRACTICALLY IMPERVIOUS
ORGANIC CLAYS OF HIGH PLASTICITY	OH	IMPERVIOUS	POOR	HIGH	POOR	LOW	MODERATE	NEGLIGIBLE TO SLIGHT	PRACTICALLY IMPERVIOUS
PEAT AND OTHER HIGHLY ORGANIC SOILS	Pt	—	—	—	—	LOW	SEVERE	—	FAIR TO POOR

ENGINEERING PROPERTIES OF SOILS

Chapter 2:

DRAINAGE

(Blank Page)

2 Drainage

2.1 CLASSIFICATION AND FUNCTION OF DRAINAGE SYSTEMS

Efficient road surface and internal pavement drainage, as well as groundwater control and slope drainage is essential for improved pavement performance. Surface drainage characteristics are influenced by road crown and crossfall, and surface layer permeability. Internal pavement structure drainage properties are influenced by the permeability of the surface layers and granular materials, and by the subgrade soil type and crossfall. Groundwater and slope drainage are concerns when discharge within the road alignment could influence slope stability or subgrade conditions. Figure 2.1.1 illustrates the various sources of infiltration into a pavement structure.

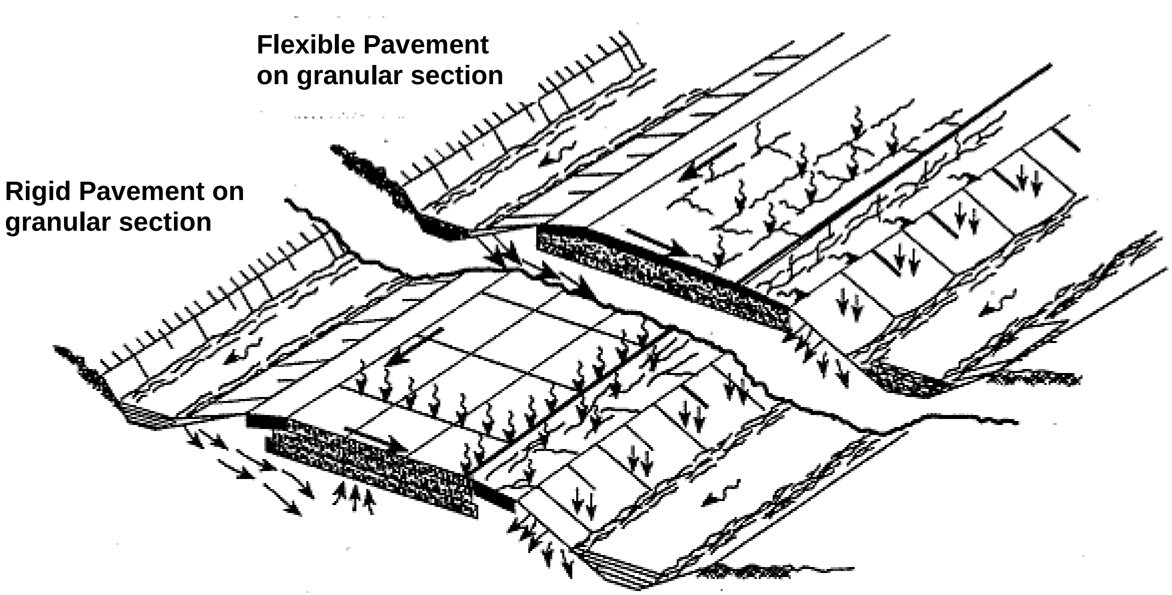


Figure 2.1.1 Sources of Water Infiltration into Flexible & Rigid Pavement Sections

2.1.1 SOURCES OF WATER

The sources of water in a pavement structure are mainly surface infiltration and groundwater. The majority of surface infiltration takes place through cracks in the pavement. Infiltration also occurs through granular shoulders and mainly contributes to pavement edge distress. Groundwater seepage into the pavement structure can result from a natural spring, a high water table rising to pavement level, or capillary action within the subgrade. Capillary rise is greatest in fine grain silty, or very fine, sandy soils.

Soil investigations should identify the position of the groundwater table relative to the elevation of the proposed road. In the design stage, the potential changes to the groundwater regime caused by the new construction should be assessed. For example, a proposed cut slope or subgrade alignment may be subjected to erosion or instability if the groundwater table is intercepted as illustrated in Figure 2.1.2.

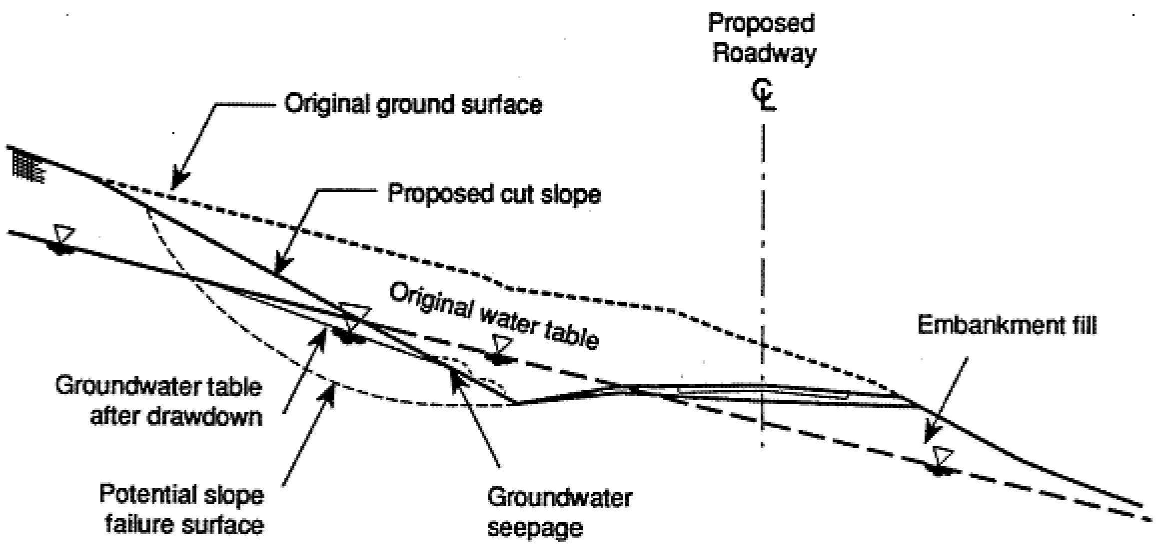


Figure 2.1.2 Potential Unstable Cut Slope due to Uncontrolled Groundwater Flow

Alternatively, the placement of fill on a slope may block the natural drainage path and cause the groundwater table to rise, as illustrated in Figure 2.1.3. Therefore, where there are large cuts or fills, the drainage system design should be carefully considered in terms of the effects on the overall road maintenance requirements, and the pavement structure performance.

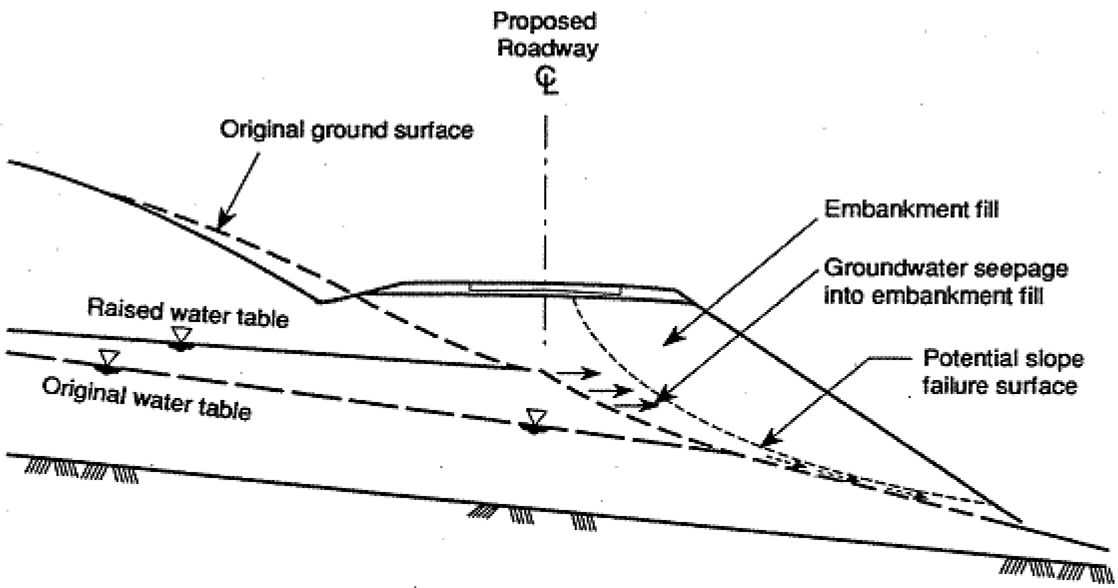


Figure 2.1.3 Potential Unstable Fill Slope due to Blockage of Groundwater Flow

This chapter summarizes the major aspects of pavement drainage. More detailed information is available in the MTO Drainage Management Manual [1] and References [2], [3] and [4].

2.1.2 SURFACE DRAINAGE

Efficient surface drainage improves the removal of water from the pavement surface to minimize surface water infiltration into the structure and to reduce the risk of hydroplaning. On most road sections this is accomplished by providing a minimum crossfall of 2% from the pavement centreline towards the pavement edge. Partially paved shoulders are used on some roads to prevent infiltration near the travelled edge of the road; however, their primary functions are safety and reduction of maintenance. Ministry experience has indicated that when partially paved shoulders are not used, increased cracking occurs at the pavement edge. Fully paved shoulders should be sloped at 4% towards the edge. Granular shoulders and partially paved shoulders should be constructed at a 6% slope.

Specified grading sections for ministry roads are shown on the Ontario Provincial Specification Drawings (OPSD) [4]. Refer to Section 3.1.6 for paved shoulder mix requirements. More details on crossfall for shoulders can be found in MTO Highway Drainage Design Standard [5]. The minimum drainage standards for pavement cross-fall requirements are provided in MTO Highway Drainage Design Standard SD-2 "Longitudinal Grade and Cross Fall" [5].

Table 2.1.1 Minimum Pavement Design Cross-fall

Minimum Design Cross-fall (m/m)	
Traffic Lanes	
Concrete or Bituminous pavement	0.02
Gravel or crushed stone	0.03 to 0.04
Shoulders[1]	
Paved or treated	0.04
Gravel or crushed stone	0.06
Earth or turf	0.08
Grassed Areas	0.04
Sidewalks	0.02
Notes: 1) On the high side of a super elevated section, the minimum cross-fall shall be 0.02 m/m.	

Surface infiltration is also minimized by the use of low permeability surface course materials. Normal asphaltic concrete and hydraulic cement concrete generally fulfill this function provided that cracks and joints in the pavement are properly sealed.

Open Graded Pavement

Open Graded Friction Course (OGFC) and other open graded asphaltic concrete pavements have a high permeability and are designed to drain water laterally through the surface course toward the pavement edge. In order to prevent the damming of water at the edge of gutters, the surface course is designed to be placed above the top of gutter. In addition, the paved edges of OGFC pavements are terminated approximately 150 mm from any vertical edge barriers.

Permeable Pavements

Permeable pavements are designed to allow percolation and infiltration of storm water into the subgrade to reduce surface run-off and promote groundwater recharge. Permeable pavements may be pervious concrete, asphaltic mixtures containing little or no fine aggregate. They also include interlocking concrete pavers founded on very coarse, uniformly graded aggregate base and sub-base. They are highly permeable systems with up to 20% voids. Due to its porous nature, there are concerns with clogging of permeable pavements over the long term from debris and winter sanding operations. Permeable pavements are restricted to low volume traffic areas and are appropriate for parking lots, patrol yards and roadway shoulders.

Earth Embankments

Surface drainage of large earth embankments needs to be considered during design where a significant quantity of sheet flow is anticipated. Mid-height berms and benches as well as interceptor ditches at the top or mid-height of slopes are effective measures for controlling sheet flow and mitigating surficial slope failures. Rip-rap lined spillways should be used to collect and convey surface water from berms and interceptor ditches down the embankment to roadside ditches.

2.1.3 INTERNAL DRAINAGE

The function of internal pavement drainage is to collect and discharge water which may enter the pavement structure through the surface course, surface cracks, and granular shoulders or from the subgrade. Internal drainage prevents the build-up of moisture which could adversely affect the durability of pavement materials as well as the strength and stability of the granular layers and subgrade. The granular layer materials are selected to promote drainage and the pavement layers are sloped away from the crown of the road. The subgrade crossfall is sloped at 3% for most types of road sections. Grading sections are shown in the OPSD drawings in Reference [4].

The drainage system may also include subdrains, granular sheeting or open-graded drainage layers (OGDL) to improve the rate at which water is collected and discharged from the roadway structure.

Subdrains may also be installed on existing roads where drainage problems have been identified. Although the ministry no longer constructs roads with earth shoulders (i.e., granular subbase ends at pavement edge), they have been built in the past and are part of the existing road network. Since the earth shoulder blocks the normal path of drainage toward the road edge, subdrains are frequently installed along the pavement edge to collect and remove excess water. Where drainage problems are encountered on earth shoulder roads without edge subdrains, drainage improvements with subdrains are generally more economical than removing the earth shoulder. Subdrain installations at pavement edges are also being carried out on concrete roads to prevent subgrade pumping at joints and cracks.

Additional information on subdrainage is provided in the MTO Drainage Management Manual [1], MTO Highway Drainage Design Standards [5] and Section 2.3 of this Manual.

2.1.4 GROUNDWATER CONTROL AND SLOPE DRAINAGE

The function of groundwater control and slope drainage is to intercept and collect groundwater in order to eliminate subgrade or slope stability problems. Difficulties with the groundwater table are encountered when cut slopes are made below the groundwater table. Slope instability may occur if the groundwater table intersects the new slope. The stability of the slope can also be at risk if surface erosion is allowed to progress to a significant level. Silty or sandy slopes are particularly susceptible to erosion. The design should consider the potential for both short-term and long-term problems.

The water table within a slope may be lowered by installing ditches or interceptor drains along the slope face or at the edge of the road. The depth of ditches or drains should be sufficient to lower the groundwater table below the face of the slope and subbase of the road. Figure 2.1.4 illustrates the function of the drainage system in controlling slope groundwater conditions.

Additional information on groundwater control and slope drainage is provided in the MTO Drainage Management Manual [1], MTO Highway Drainage Design Standard [5] and Section 2.4 of this Manual.

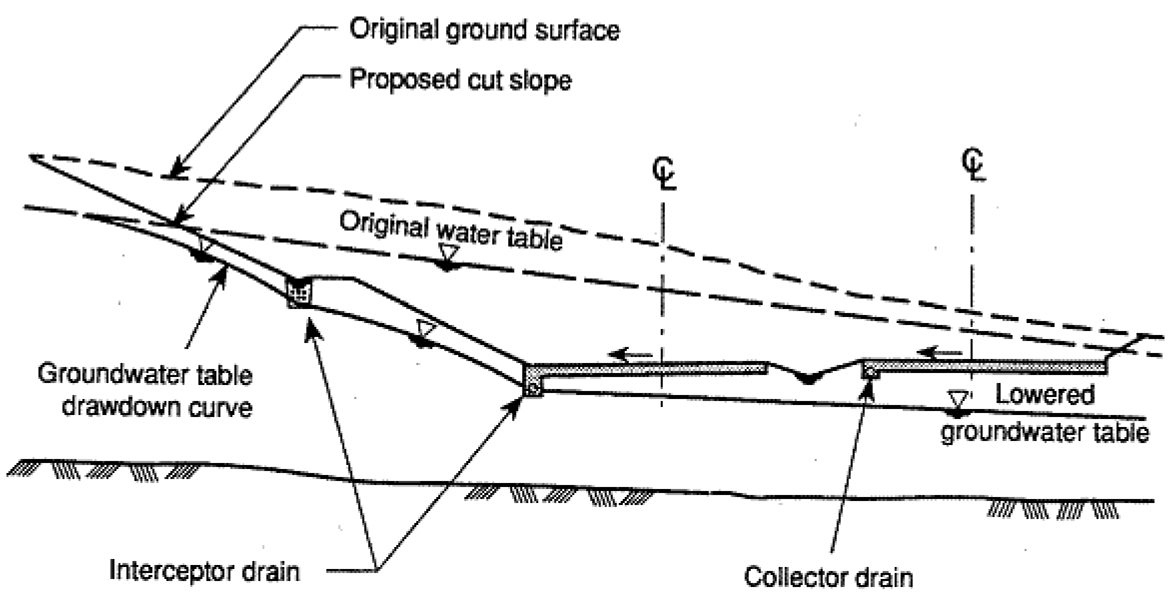


Figure 2.1.4 Illustration of Multiple Longitudinal Drain Installations in Controlling Pavement and Slope Drainage

2.2 EFFECTS OF WATER ON PAVEMENT STRUCTURE

The presence of water in a pavement structure can affect its structural and functional properties. These effects can be subdivided into four general categories: frost action; material strength; rigid pavement joint performance; and durability of bonded materials.

2.2.1 FROST ACTION

Distress manifestations associated with frost action include frost heaving and frost boils. Fine grained silty soils and silty granular materials are subject to frost action in the presence of moisture and freezing temperatures. Water can be drawn up to the freezing front by thermodynamic forces and capillary action to form ice lenses in frost-susceptible soils. These conditions are aggravated when the water table is at a shallow depth. Pavement structures are generally weakest in the spring, irrespective of the subgrade soil type, due to the higher moisture regime which is generated by the melting of ice lenses and increased precipitation and infiltration rates. The higher moisture regime results in reduced strength properties in the granular layers and subgrade. The weakest pavement condition normally occurs when all layers are thawed and the subgrade is saturated and has low support properties. This frost action and loss of support contributes to the formation of potholes.

2.2.2 MATERIAL STRENGTH

The presence of water within the pavement structure reduces the strength of both the granular layers and the subgrade, resulting in a lack of support for the overlying pavement. The loss of strength is caused by the generation of excess soil pore water pressures when a moving wheel loads the pavement. The development of high pore water pressures prevents the full frictional capacity of the granular or subgrade from being mobilized as it would be if it were in a dry or moist condition. Figure 2.2.1 illustrates the effects of saturated granular base conditions on flexible pavement performance.

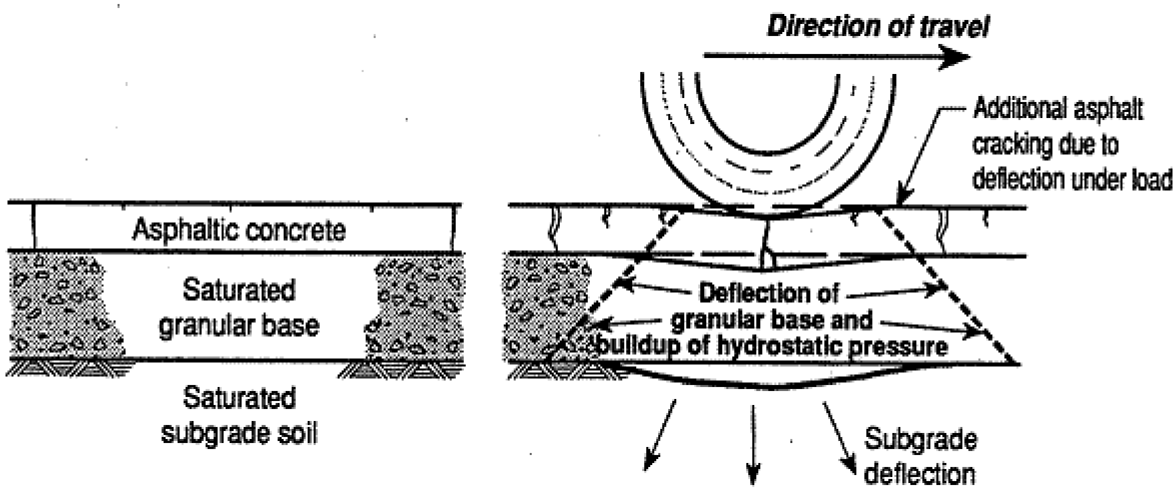


Figure 2.2.1 Influence of Saturated Granular Base and Subgrade on Flexible Pavement Performance under Dynamic Loading

2.2.3 RIGID PAVEMENT JOINT PERFORMANCE

Water contributes to stepping (faulting) of joints and transverse cracks in a rigid pavement structure. Figure 2.2.2 illustrates rigid pavement joint performance under a moving vehicle. The action of a wheel passing from one slab to the next results in a slurry of water and fines being forced out from the edge of the approach slab. As the wheel transfers from the approach slab to the leave slab, fines and water from under the leave slab are moved by suction forces to fill the void under the approach slab. Continued wheel load repetitions in the presence of water result in the edge of the approach slab progressively heaving as the leading edge of the leave slab settles, which results in a stepped joint or crack, and eventual loss of support and slab cracking. The provision of granular and permeable bases under a rigid pavement is essential for effective subsurface drainage and the performance of the pavement structure.

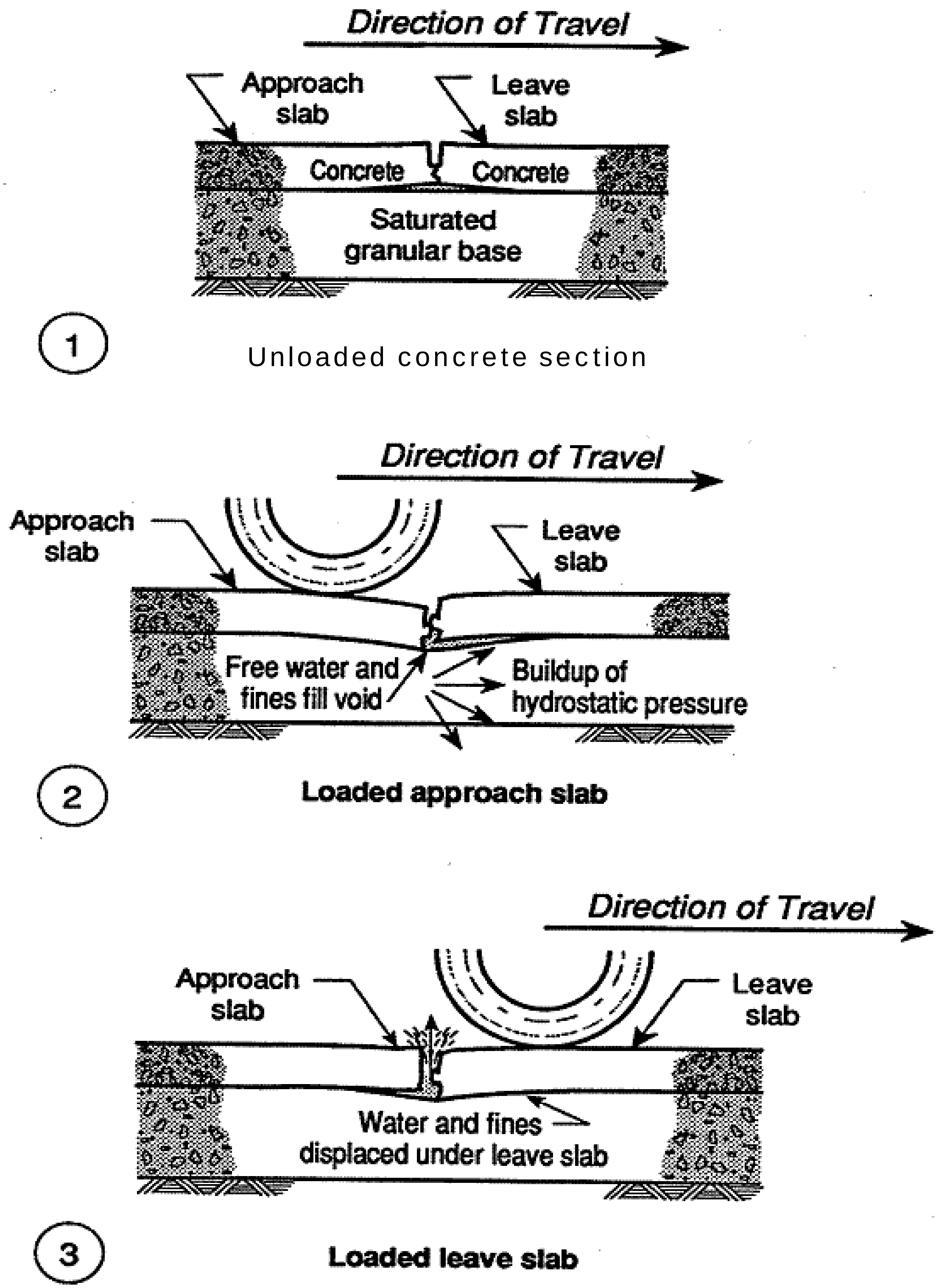


Figure 2.2.2 Concrete Joint Faulting due to Saturated Granular Base and Dynamic Loading Conditions

2.2.4 DURABILITY OF BONDED MATERIALS

Bonded pavement layers include asphaltic concrete, surface treatments, hydraulic (Portland) cement concrete, and various treated bases. The durability of these materials is influenced by exposure to excess moisture. Asphaltic concrete and surface treated aggregates may be susceptible to "stripping" due to the effects of water causing bond failure between the asphalt cement and aggregate surface. De-icing salts in solution can be damaging to concrete. Ministry construction materials are specified to have properties which are resistant to deterioration from the effects of water and water transported chemicals.

2.3 DESIGN OF DRAINAGE SYSTEMS FOR PAVEMENT STRUCTURES

2.3.1 RURAL AND URBAN DRAINAGE SYSTEMS

The majority of ministry rural roads use ditches to collect and remove excess surface water. In earth cut sections the ditch is located directly adjacent to the road and the ditch invert must be at least 0.5 m below the top of subgrade. In fill sections the ditch invert should extend at least 0.25 m below the base of the fill and should be separated from the toe of the fill by a 1.5 m plateau. Specified ditching sections are provided in the 200 series of the OPSD [4].

The ditch system is almost exclusively used in rural areas where there are usually no space constraints and storm sewer systems. In urban areas, the drainage system usually controlled edge drainage. The pavement edge drainage consists of curbs and catchbasins with subdrains placed at the edge of pavement.

Uniform grades with positive slopes are extremely important to the proper performance of the selected drainage system. Design grades should ensure that there are no low points within the pavement structure where water can stand.

2.3.2 OPEN-GRADED DRAINAGE LAYERS

Rapid and effective drainage of the pavement structure is critical in minimizing distress associated with entrapped water. Free-draining designs can be best achieved by placing a highly permeable open-graded drainage layer (OGDL) beneath the pavement wearing surface. A typical cross-section where an OGDL has been used is illustrated in Figure 2.3.1.

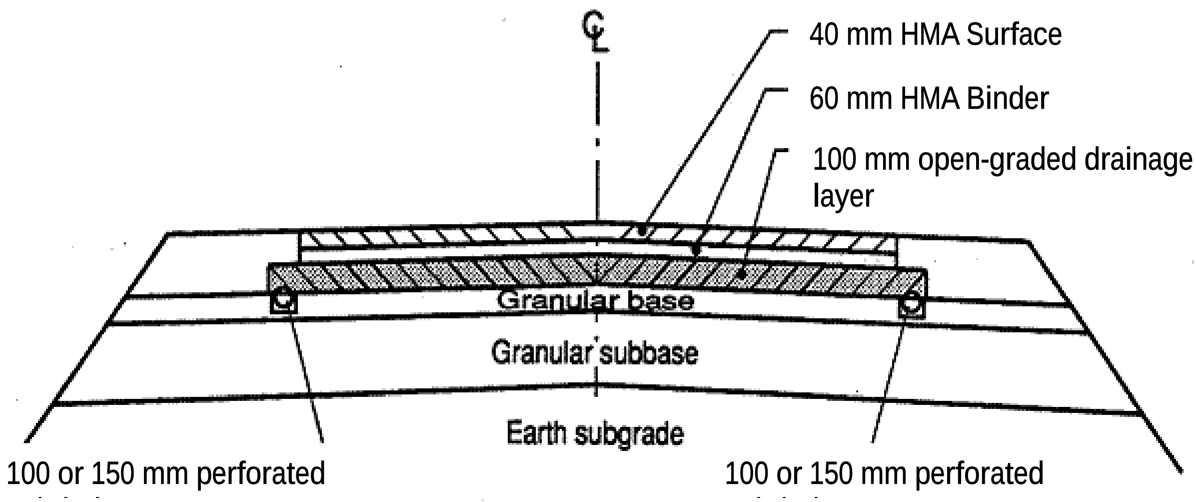
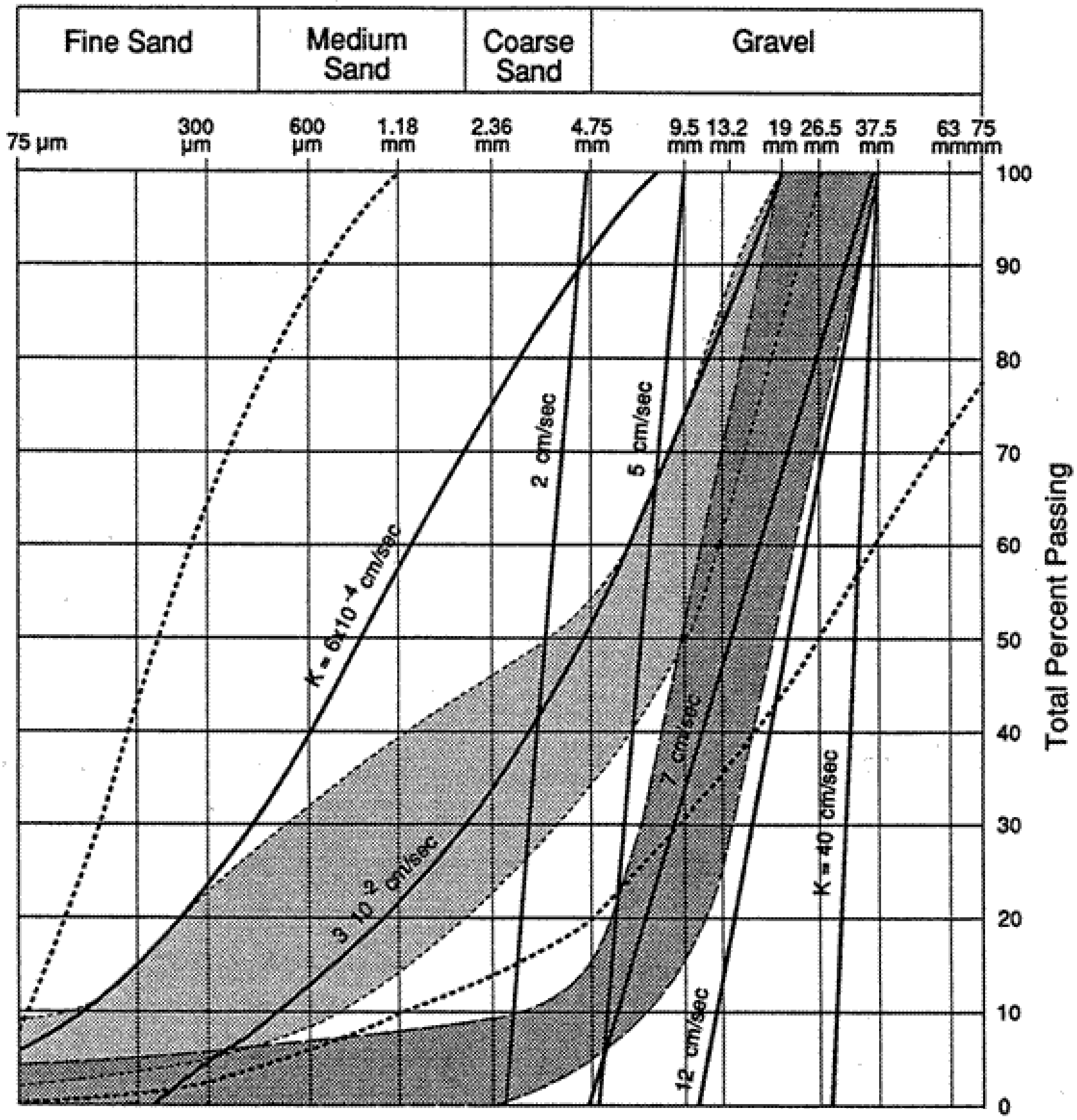


Figure 2.3.1 Cross-Section of Typical Pavement with an Open-Graded Drainage Layer

Traditionally, the ministry has assumed that the granular base and subbase materials are "free-draining". However due to high fine content, the permeabilities of these materials can be low. Figure 2.3.2 illustrates gradation bands of ministry specified base, subbase and OGDL materials along with typical permabilities (k, cm/s).



MTO Standard Sieve Sizes

Legend:

Granular Base K = 10^{-3} to 10^{-5} cm/sec

Granular Subbase K = 10^{-2} to 10^{-6} cm/sec

OGDL Gradation Limits K = 10 to 10^{-1} cm/sec

Figure 2.3.2 Base, Subbase and OGDL Permeability and Gradation Plot

OGDL as specified by the ministry is essentially an open graded free draining product. Stabilized OGDL may be treated with 1.5 to 2.0% asphalt cement or 120 to 180 kg of hydraulic cement per tonne [6]. OGDL is placed using conventional paving equipment in a single 100 mm lift normally on a minimum 150 mm to 300 mm thick layer of Granular A, which acts as a filter to prevent subgrade fines from migrating upwards and contaminating the OGDL. Drainage is achieved by grading the OGDL at 2% (minimum) towards the pavement shoulder and outletting in a collector system. A normal compaction sequence of rollers is specified; however construction traffic is prohibited from travelling on the OGDL except for paving or slipforming operations.

Granular O may be used as a drainage layer on freeway projects below a concrete base. Granular O is a well-graded material with 100% of the particles passing the 37.5 mm sieve size, and no greater than 5% material passing the 75 μm sieve size.

Currently, OGDL is specified for all rigid and composite pavement designs, and considered for all new flexible pavement designs.

2.3.3 SUBSURFACE COLLECTOR SYSTEMS

Collector systems consist of subdrains, French drains, and other pre-manufactured systems placed under a roadway or within its right-of-way to collect and remove water within the pavement structure. Collector systems used on slopes are described in Section 2.4. In this section, the collector systems used under or directly adjacent to the pavement are discussed.

Subdrains consisting of flexible perforated or slotted plastic pipe (commonly 100 or 150 mm in diameter) wrapped with a geotextile are the most common type of collector system used by the ministry. Subdrains are installed in localized wet areas, in existing pavements with drainage problems, or at the edge of pavements on new or existing roads to improve drainage. The drains are normally installed in shallow, narrow trenches, to ensure that the pavement granular layers are positively drained. Laterals outletting the subdrains are not perforated, and the last 2.5 m consists of corrugated steel pipe equipped with a rodent gate. Further information on subdrainage systems is available in References [7] and [8].

The selection of free-draining granular material around the subdrain pipe depends on the expected quantity of water as well as on the availability of materials. Where large quantities of water are anticipated the pipe should be surrounded by clear stone to ensure more efficient collection and removal. A geotextile wrapping should be used to prevent the drain from clogging. The selection and location of the geotextile depends on the gradation of the subgrade and the granular material. the geotextile fabric should be wrapped around the outside of the clear stone to create a French drain. Plastic pipe supplied with a knitted filter sock geotextile are commercially available. The choice of a geotextile depends on the gradation of the subgrade and granular, as discussed in Section 1.6.3 of this Manual.

Subdrains are normally installed at the edge of pavement or edge of shoulder. Where the shoulders are granular, it is preferable to place the drains directly under the edge of pavement to avoid infiltration of surface water.

2.3.4 FILTER DESIGN

Drainage systems used by the ministry normally require filter protection from clogging by fines migrating from adjacent granular and subgrade soils. Subdrain pipes as well as granular drainage systems may be subject to clogging and silting and the potential for these problems should be reviewed during the design stage.

Where possible, granular materials used in ministry drainage systems should be chosen to have adequate permeability and filtering capability to prevent the migration of fines from adjacent soils. Design criteria have been developed relating the gradation of the granular material to the adjacent soil to ensure that both requirements can be achieved.

To ensure adequate permeability:

$$\frac{D_{15}\ \text{Filter (grain size at 15\% passing)}}{D_{15}\ \text{Subsoil}} > 5$$

To prevent migration of soil fines:

$$\frac{D_{15}\ \text{Filter}}{D_{85}\ \text{subsoil}} < 5$$

Normally, concrete sand or sandy Granular B are suitable filter materials. Concrete sand or Granular B placed next to most subgrade soils will provide adequate drainage as well as prevent the migration of fines. These properties are discussed in more detail in MTO Drainage Management Manual [1].

Where a filter layer provides adequate permeability but does not prevent the migration of fines, an intermediate filter layer must be placed. Geotextiles are used exclusively in subdrains or interceptor drains to prevent clogging of the pipe and loss of soil. Geotextiles can also be used under rip-rap in ditches, behind reinforced gabion walls, and at drain and culvert outlets to prevent the erosion of underlying subsoil.

The selection of geotextiles with respect to filtering capacity is discussed in Section 1.6.3 of this Manual and in the referenced ministry publications.

2.4 GROUNDWATER CONTROL AND SOIL SLOPE DRAINAGE SYSTEMS

The position of the groundwater table with an earth slope has a significant effect on its stability. Water reduces the frictional strength of a soil and will reduce the stability of the slope. If the water table exits from the face of a slope, erosion is a concern as well. Slope stability can be improved by drainage and a number of alternative drainage systems are available for consideration in design.

2.4.1 INTERCEPTOR DRAINS

Interceptor drains are placed along the horizontal plane of the slope face in order to lower the groundwater table (Figure 2.1.4). An interceptor drain improves slope stability, prevents erosion by lowering the water table below the slope face, and improves pavement performance by lowering the water table below the pavement structure. If the slope is small, a single line of drains may be sufficient to adequately lower the groundwater table. On longer and higher slopes several lines of drains may be necessary, including an interceptor drain below the ditch beside the road.

Drains normally consist of a 100 or 150 mm diameter perforated, flexible plastic pipe surrounded by coarse granular material placed in a trench. Alternatively, a French drain type design consisting of clear stone alone may be used. The depth of the drain and its position on the slope is dependent on how much the water table has to be lowered. Both the coarse aggregate and the pipe may have to be protected from clogging with fine soil particles by using a geotextile. The geotextile should completely surround the aggregate and pipe. Additional information on geotextile design parameters are provided in Section 1.6.3 of this Manual.

The depth and positioning of the drain is critical to the success of the groundwater control program. Where difficult problems are encountered requiring a sophisticated analysis, the appropriate head office unit should be contacted as discussed in Section 1.1.4.

2.4.2 PARALLEL DRAINS

A parallel drain system consists of perforated or slotted pipe surrounded by aggregate placed in a grid or herring bone pattern on the slope face. The parallel drains are placed in shallow trenches excavated into the slope face.

2.4.3 GRANULAR SHEETING

Granular sheeting is normally placed on cut slopes in erodible soils to provide a stable, free-draining, slope face. The granular sheeting consists of a 300 to 600 mm thick gravel layer or crushed stone which is comprised of clean and slightly coarser grading granular. The treatment is particularly suitable where groundwater discharge through to the surface of the slope face will contribute to erosion. The gradation of granular sheeting is specified in OPSS 1004.

2.4.4 COUNTERFORT DRAIN

A counterfort drain is a drain which runs down the face of a slope or embankment and is used to lower the water table. The drain normally consists of a 100 or 150 mm diameter flexible plastic pipe surrounded by granular material placed in a trench excavated into the slope. If clear stone is used, the pipe may be eliminated. Either the clear stone or the pipe may have to be protected with geotextile to prevent clogging.

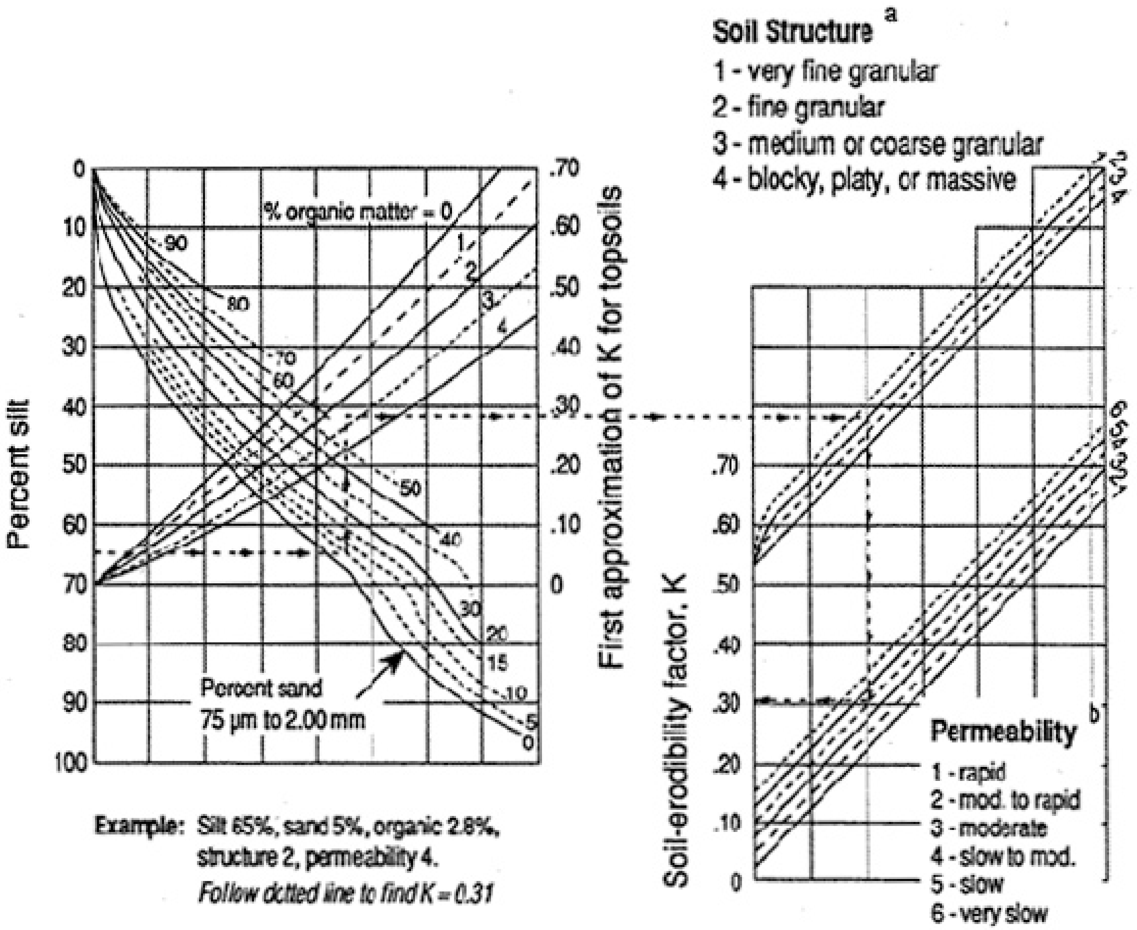
2.4.5 DRAINAGE BLANKET UNDER PAVEMENT STRUCTURES

A drainage blanket is a layer of highly permeable granular which is placed beneath the pavement structure where a road is constructed over a spring or groundwater discharge area. Parallel drain achieves the same objective, although the blanket drain provides more uniform coverage and drainage capability. The blanket drain is sloped towards a ditch or subdrains installed at the edge of the road to provide a positive outlet. Clear stone is used in this application. Most drainage blankets should be sandwiched between geotextile to prevent (i) subgrade fines from moving upwards into the blanket, and (ii) subbase fines from moving downwards into the blanket.

2.5 PREDICTION OF SOIL ERODIBILITY

Soil erosion is the removal and transport of soil particles. Progressive removal may result in the undermining of a road, instability of a cut slope or loss of property. Sedimentation of the soil particles may cause blockage of ditches and sewers, environmental damage to wet lands and bodies of water, or crop damage. Therefore, it is important that the potential for erosion be properly assessed when designing a road. Erosion is affected by a number of factors including rainfall intensity, slope geometry, vegetation cover, and the soil type. Erosion potential during the construction phase and in the long term must be considered, and the use of erosion control blankets and mid-height berms or benches recommended for situations where significant erosion is anticipated.

The erosion potential of soil is influenced by the clay, silt, sand and organic content, as well as the soil structure and the soil permeability. The Wischmeier Nomograph shown in Figure 2.5.1 relates soil erodibility to these soil properties and the analysis method is described in detail in the MTO Drainage Management Manual [1]. The Wischmeier Nomograph generates a Factor, K, with a value between 0 and 1.0, which categorizes the erodibility of the soil. The higher the value of K, the greater the erodibility of the soil. Highly erodible silty soils may have a K factor exceeding 0.6, while relatively non-erodible soils may have a factor less than 0.2. The value of K increases as the silt and very fine sand content increases; the organic content decreases, and the permeability decreases. Small clay content does not assist in reducing erosion since it reduces soil permeability and increases runoff. High clay content increases the cohesion of the soil and, although the runoff is still high, acts to reduce erosion.



a. **Soil Structure Equivalents**

1 - Very fine granular	Very fine sand
2 - Fine granular	Fine or medium sand
3 - Medium or coarse granular	Coarse sand or gravel
4 - Blocky, platy, or massive	Clay

b. **Permeability Equivalents**

1 - Rapid	Gravel
2 - Rapid to moderate	Sand
3 - Moderate	Silty sand to sandy silt
4 - Moderate to slow	Clayey silt
5 - Slow	Silty clay
6 - Very slow	Light to heavy clay

Figure 2.5.1 Wischmeier Nomograph

The potential for erosion should also be considered at all subdrain and culvert outlets. The use of rip-rap overlying geotextile at these locations should be assessed. The selection of the type of geotextile depends on the soil type, as discussed in Section 2.3.4 and Section 1.6.3.

2.6 MAINTENANCE

Maintenance of drainage systems is discussed in detail in the MTO Drainage Management Manual [1], and the information is briefly outlined in this Manual. The most critical maintenance elements of pavement drainage systems are the clearing of subdrain outlets and ditches, and crack sealing of hot mix asphalt (HMA) pavements. Additional information on crack sealing is provided in Section 4.2.1.

Outlets can become clogged by vegetation, debris and sedimentation. The inspection of ditches and outlets is covered in - Maintenance Quality Standard MQS-501, and MQS-502, MTO Maintenance Manual 2003. Outlets are to be inspected twice a year, once in the fall and the following spring at the start of the thaw period. Outlets are located by survey chainage taken during construction and by marker posts placed next to the outlets. Marker posts consist of a 25 x 25 mm square galvanized steel bar, embedded into the ground, adjacent to the outlet, clearly visible from the driving portion of the roadway. Ditches should be checked on a regular basis to ensure that they are clear of debris.

Crack sealing early in the life of a pavement minimizes the infiltration of surface water into the pavement structure and can significantly extend the life of a pavement. Same thing applies to joint re-sealing of concrete pavements. A timely crack sealing and joint re-sealing program can reduce the rate of pavement deterioration due to surface water infiltration and therefore represents an economical method of pavement maintenance. The pavement surface should be examined on a yearly basis to identify cracking.

(Blank Page)

REFERENCES TO CHAPTER 2

[1] Ministry of Transportation of Ontario, *MTO Drainage Management Manual*, 1997

[2] Moulton, L.K., *Highway Subdrainage Design*, U.S. Department of Transportation Federal Highway Administration, Report No. FHWA - TS-80-224, August, 1980

[3] Transportation Research Board, *Pavement Subsurface Drainage Systems*, National Cooperative Highway Research Program, NCHRP of Highway Practice 96, November, 1997.

[4] Ministry of Transportation of Ontario, *Ontario Provincial Standards for Roads and Public Works, OPS Volume 3 – Drawings for Roads, Drainage, Sanitary, Sewers, Watermains and Structures*.

[5] Ministry of Transportation of Ontario, *Highway Drainage Design Standards,* January 2008.

[6] Transportation Association of Canada, *Pavement Design and Management Guide*, TAC, Ottawa Canada, 1997.

[7] McMaster, B., *Study On Improving Pavement Drainage Using Plastic Pipe Subdrains*, Ministry of Transportation and Communications, Engineering Materials Office, report EM10, 1978.

[8] McMaster, B., *Improving Pavement Drainage Using Plastic Pipe Subdrains Update*, Ministry of Transportation and Communications, Engineering Materials Office, report MI-17, 1979.

Chapter 3:

PAVEMENT DESIGN

(Blank Page)

3 Pavement Design

3.1 INTRODUCTION

The objective of pavement design is to develop a cost-effective pavement structure that addresses site specific performance, serviceability and safety requirements. It requires an understanding of soils and paving materials and their behavior under different traffic and climatic loading conditions. Pavement design is not an exact science and there are many variables that influence pavement performance making the analysis complex. This chapter discusses the engineering principles used in the design of pavement.

All pavement designs are influenced by the following factors to a greater or lesser degree as illustrated in Figure 3.1.1 [1]:

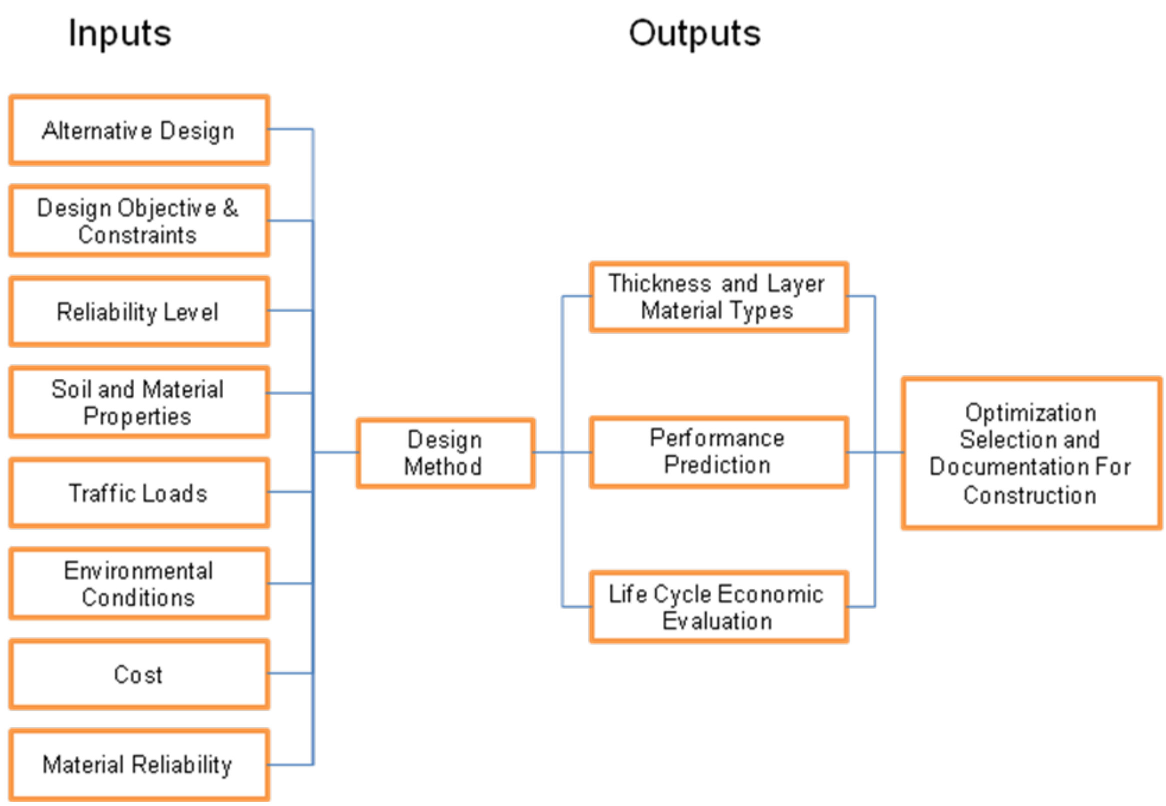


Figure 3.1.1 Functional Scope of Pavement Design

Traffic loading is one of the most important pavement design factors and includes traffic volume, growth rates, axle loads and distribution, tire pressures and vehicle suspension characteristics. Environmental considerations include precipitation, moisture in pavement layers, temperature ranges, and freeze-thaw cycles. Subgrade soil type, moisture content and other physical properties also have a significant influence to the pavement design.

3.2 DESIGN CONSIDERATIONS

3.2.1 GEOMETRIC

The geometric features of highway design are covered in the Geometric Design Standards for Ontario Highways [2]. The design elements are based on speed, traffic volumes and existing constraints.

A Scope and Cost Report [3] is required for each MTO design project prior to being placed on a 5-year program plan. Preliminary pavement designs are developed based on pavement management system (PMS-2) data, historical data and other existing reports.

A Design Criteria is prepared for each project as described in PHY Directive B-220. The Design Criteria typically includes traffic information and the scope of work. The Design Criteria identify the limits of the project, the existing conditions and governing geometric standards, and provides details about drainage, pavement rehabilitation strategies and construction staging, etc. Table 3.2.1 gives the typical summary of the Design Criteria.

Table 3.2.1 Typical Information on Design Criteria

	PRESENT CONDITIONS	DESIGN STANDARDS	PROPOSED STANDARDS
HIGHWAY CLASSIFICATION	RAU 100	RAU 100	RAU 100 [a]
MIN STOPPING SIGHT DISTANCE	120 m	185 m	120 m [b]
EQUIVALENT MIN 'K' FACTOR - Crest	41	70	41 [c]
EQUIVALENT MIN 'K' FACTOR - Sag	55	45	55
GRADES MAXIMUM	3.2 %	6 - 8%	3.2 %
MINIMUM RADIUS	318 m	420 m	318 m [d]
PAVEMENT (LANE) WIDTH	3.35 to 3.94 m (Varies)	3.75 m	3.5 - 3.75 (Varies) [e]
SHOULDER WIDTH (Outside)	1.8 to 5.6 m Varies	3.0 m	Varies [f]
SHOULDER ROUNDING	Varies	0.5 m	0.5 m
R.O.W. WIDTH	14 to 35 m (Varies)	-	14 to 35 m (Varies) [g]
POSTED SPEED	50 to 80 km/h (Varies)	80 km/h	50 to 80 km/h (Varies) [a]

3.2.2 PAVEMENT COMPONENTS

Definition of Pavement Structure

Conventional pavement structure consists of layers of subbase (normally Granular B), base (normally Granular A) and surface course. The base layer usually is Granular B, and base layer usually is Granular A. The surface course could be either asphalt material or concrete material. These layers are founded on a subgrade having sufficient strength to support the traffic load and distribute it across the roadbed.

The pavement surface is that part of the pavement structure above the base course. It includes a layer or layers of specified materials, normally asphaltic concrete (hot mix asphalt), hydraulic (Portland) cement concrete, plant or in-place processing, microsurfacing or surface treatments. In asphaltic concrete pavements, the upper layer of the pavement surface is the wearing or surface course, and it provides a durable, smooth and skid resistant surface. Asphaltic concrete layers beneath this wearing course are called "binder courses" [4], [5].

Depending upon local conditions and economics, the granular subbase and/or base may be omitted or the granular subbase and/or base may be supplemented or replaced with cement treated, asphalt cement treated or lean concrete bases or an open-graded drainage layer. In composite pavements, the concrete base forms part of the pavement. The subgrade may also be improved when stabilized with materials such as lime and fly ash.

Major Functions and Characteristics of Pavement Structure Layers

Surface Course - Generally a dense, highly stable, durable, skid-resistant surface course of hot mix asphalt carries the traffic. It normally contains the highest quality selection of materials to meet the requirements for such characteristics as friction, smoothness, noise control, rutting and shoving resistance and drainage. Typically, the surface course is designed to be impermeable to keep water out of the underlying pavement structure, but some urban freeways incorporate an open-graded friction course designed specifically to provide low traffic noise transmission characteristics [5].

A concrete pavement surface also distributes load stresses in addition to acting as a wearing surface.

Binder Course - The binder course is the lower layer(s) of an asphaltic concrete pavement. It is normally distinguished from the surface course when there is a distinct difference in the quality of the mixtures used. It adds to the overall strength of the pavement structure by supporting the surface course and distributing load to the base.

Granular Base Course - The granular base course carries a large portion of the load, provides drainage, minimizes erosion, minimizes frost action and allows fine tolerance in grading to give a level surface upon which to lay the asphalt or concrete pavement. It is generally composed of high-stability graded crushed gravel or stone [6].

Granular Subbase Course - This is normally a non-processed material obtained from local gravel or sand pits. It may be a processed material from quarries or crushed bedrock sources if higher stabilities are required or where gravel sources are unavailable.

Granular subbase performs the same function as a granular base course except that it may be of lesser quality. It provides the working platform for pavement construction.

Subgrade - This is the foundation for both flexible and rigid pavement structures. Subgrade soil consists of both native soil remaining after the removal of top layers, as well as soil material used to construct elevated highway sections (fills). It carries the pavement structure and a portion of wheel load stresses.

Pavement Surface Types and Shoulder

A surface course mix type is selected on the basis of its wearing resistance, durability and friction properties. On King's Highways and secondary paved highways in Ontario, these hot mix asphalt (HMA) surface courses, referred to as SuperPave 12.5 FC2, SuperPave 12.5 FC1, SuperPave 12.5 and SMA 12.5, are also designated as HMA surface course type mixes and they are generally used in rural and urban freeways. Where appropriate, concrete surfaces or alternative surfaces may also be considered.

Traffic volumes (present and projected) are the main factor used in determining surface course hot mix types. Other factors, such as posted speed, type and percentage of trucks, accident rate, highway classification, noise levels, and continuity should be considered.

Ministry surface course directive (MTO PHM-C-001) identifies, for each region, those highways that are required to have SuperPave or SMA as the surface course hot mix type [7]. King's highways not indicated will normally have SuperPave 12.5 as the surface course mix type.

Secondary highways will normally have either SuperPave 12.5, or a surface treatment or granular surface, depending on the function of the road. Refer to Section 3.5 for Low Volume Road Designs.

Partially and Fully Paved Shoulder

Partially paved and fully paved shoulders are warranted on many sections of highway for safety and erosion control purposes. The policy is included in the Geometric Design Standard of Ontario Highways [2]. All major capital construction, reconstruction or pavement rehabilitation should include fully paved shoulders. For more details, refer to the memorandum for fully paved shoulder on freeways released in 2001.

An important consideration in the design, construction, and performance of the partially paved shoulder is the avoidance of a joint at the interface of the pavement /partially paved shoulder. For this reason, partially paved shoulders should be constructed in conjunction with resurfacing or reconstruction work. Also, shoulders may be used for future traffic staging during construction.

For safety and performance, both partially paved and fully paved shoulders should be delineated by edge painting of the through lanes and/or installation of a rumble strip. MTO Directive PLNG-B-004 describes the design and installation for shoulder rumble strip. A shoulder rumble strip is a grooved formation installed within the paved shoulder or partially paved shoulder on a highway. The intention of shoulder rumble strips is to provide the motorist with both an audible and tactile warning that the vehicle has partially or completely departed the travelled way of a highway. An encounter with shoulder rumble strips is expected to alert an inattentive motorist to steer the vehicle back onto the travelled way of the highway.

When fully paved shoulders are warranted, the following shoulder mix types may be used based on the surface course selected as shown below.

Surface Course Selected	**Surface Type of Shoulder**
Superave 12.5 or Surface Treatment	SuperPave 12.5 or Surface Treatment
SuperPave 12.5 FC1 or FC2	SuperPave 12.5
SMA 12.5	SMA 12.5
Concrete	Concrete or hot mix as retrofit

Surface treatments can be considered on fully paved shoulders under specific site conditions, such as substandard shoulder widths and maintenance concerns. Usually 100% RAP (reclaimed asphalt pavement) or granular sealing are the granular shoulder treatments for rounding.

Continuity of surface type mix is very important. Care should be taken not to block internal drainage of the permeable surface course in the driving lane with impermeable shoulder mixes.

3.2.3 TRAFFIC VOLUME AND LOADINGS

Traffic generates pavement distress through fatigue and creep of materials which results in cracking and permanent deformation. Therefore, a key element of pavement design is having reliable estimates of traffic volumes and maximum axle loads. Traffic volumes provide an indication of the number of repeated loads expected over the pavement design life. Maximum axle loads are used in design to ensure that the pavement structure has sufficient strength.

This section describes the measures used for estimating traffic volumes and the characterization of axle loads and their impacts. Allowable loads governed by The Highway Traffic Act in accordance with axle load limits prescribed in the Canadian Highway Bridge Design Code (CHBCD), CAN/CSA-S6-06 [8].

Traffic Volumes

Traffic volumes on Provincial Highways are published annually in both summary and detailed form by the Traffic Office, Highway Standards Branch. The publication "Provincial Highways Traffic Volumes" available at:

http://www.mto.gov.on.ca/english/pubs/trafficvolumes.shtml

It provides traffic volumes on an annual and seasonal average basis for selected corridors of the provincial highway network.

The basic measures used to characterize these volumes are:

- AADT - Annual Average Daily Traffic
  (The average 24-hour, two-way traffic for the period January 1st to December 31st).
- SADT - Summer Average Daily Traffic
  (The average 24-hour, two-way traffic for the period July 1st to August 31st, including weekends).
- SAWDT - Summer Annual Weekday Daily Traffic
  (The same as SADT but excluding weekends, from Friday noon to Monday noon).
- WADT - Winter Average Daily Traffic

(The same as AADT but only for the period December 1st to March 31st).

Traffic volumes refer to all vehicles in the stream, including automobiles and commercial vehicles (trucks and buses). It is the proportion of the latter type, particularly the number of heavy trucks, that is of principal interest to the pavement designer.

<u>Truck Model Based on the Ontario Commercial Vehicle Survey (CVS)</u>

While traditional traffic classification data collection technologies including Weigh-In-Motion (WIM) and Automated Traffic Recorders (ATR), are accurate, they measure traffic activity at a single location only.

The Commercial Vehicle Survey (CVS) is a roadside intercept survey of truck operators conducted on a 24-hour basis every day of the week. It provides an accurate representation of trucking activities. CVS is carried out at truck inspection stations and truck lay-bys. Trucks are randomly selected from a stream of traffic by MTO Carrier Enforcement Officers. CVS captures inter-city truck travel characteristics related to the driver, vehicle, carrier, trip and commodity. Surveys are conducted at 150 sites across the province including border crossings. For example, in 2006-2007, the surveys collected information on approximately 70,000 intercity trips. This equates to 18.3 million km of truck travel per day or 82% of provincial truck activity.

The data and information from the surveys is used for provincial highway initiatives such as national border crossing studies, highway studies and environmental assessments. It also provides input data for freight policy related to lift axle regulations.

For pavement management purposes, CVS is used to replicate the total truck activity on provincial highways. Daily truck activity on each provincial link is modeled to generate total link ESALs by travel direction. The CVS application is GIS based and stored in a database format. The results are available in the form of maps upon request for specific highway sections.

Axle Loads for Pavement Design

Pavements are designed to carry axle load limits established by the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06 [8] to ensure that the as-built capacity of bridge and pavement structures are the same. Different axle groups have different load impacts on pavement; therefore it is important to identify the axle group and maximum allowable weight on each group when performing pavement design. The Ontario Bridge Formula [9] is the basis for establishing regulatory loads in Ontario.

Allowable Gross and Single Axle Loads

<u>Truck or Buses</u>

The allowable single axle load for trucks or buses in Ontario is 9 tonnes for a single axle with single tires and 10 tonnes for a single axle with dual tires. An axle is considered a single axle provided it is at a distance of 2.5 m or more from its neighbour. Two axles within one metre spacing are also considered a 'single axle'. The allowable load for dual and triple axle units varies with spacing between the axles. The allowable loads are found in tables in the Highway Traffic Act, under O.Reg. 413/05. Maximum loading for a dual axle truck is 19.1 tonnes (18 tonnes with single tires).

The maximum allowable gross weight depends upon the configuration of axles and inter-vehicle unit distance but is not to exceed 63.5 tonnes. In no case shall the tire loading exceed 9 kg/mm of tire width for tires less than 150 mm total width and 11 kg/mm for tires that are 150 mm or wider.

Tractor Trailers

The allowable single axle load for tractor trailers in Ontario is 9 tonnes for a single (non-steering) axle with single tires and 10 tonnes for a single (non-steering) axle with dual tires. The front steering axle of tractors is limited to 7.7 tonnes. The allowable load for tandem and tridem axle units varies with spacing between the axles. The allowable loads are listed under O.Reg. 413/05 of the Highway Traffic Act:
http://www.e-laws.gov.on.ca/html/regs/english/elaws_regs_050413_e.htm
The maximum loading for a tandem axle tractor trailer is 19.1 tonnes (18 tonnes with single tires) and 26 tonnes for a tridem axle.

The maximum allowable gross weight depends upon the vehicle configuration and inter-vehicle unit distance but is not to exceed 63.5 tonnes. In no case shall the tire loading exceed 9 kg/mm of tire width for tires less than 150 mm total width, 10 kg/mm for non-steering axles equipped with single tires and 11 kg/mm for all other tires.

Concept of Load Equivalency Factor

The Load Equivalency Factor (LEF) is used to characterize the spectrum of axle loads applied to a pavement. It is a relative measurement of pavement damage determined by converting mixed vehicle traffic loading into an equivalent single axle load. The standard LEF for a single axle load of 80 kN (8.2 tonnes) is 1.0, which is also referred to as Equivalent Single Axle Load (ESAL). ESAL is a means of equating various axle loads and configuration to represent the standard pavement damage of 80 kN per single axle. Axle loads greater or less than 80 kN, (8.2 tonnes) have LEF values greater or less than 1.0 applied to them respectively, in order to calculate the number of ESAL.

Load Equivalency Factor = (specific axle group load / standard axle group load)^4

LEF varies approximately according to the "4th power law". For example, a 100 kN, (10.2 tonnes) axle load would result in a LEF = $(100/80)^4 \sim 2.5$. Thus, this 25% increase in actual load would result in 250% more damage; i.e., 2.5 ESALs.

ESAL determination is not as simple as outlined above. LEF is a function of axle group type, pavement type, thickness and serviceability. The American Association of State Highway and Transportation Officials (AASHTO) differentiate between the rigid and flexible ESALs as the loss of serviceability from an ESAL. The AASHTO ESAL concept was different then the American Association of State Highway Officials (AASHO) Road Test for the two basic pavement types. Consequently, AASHTO LEF's calculations are dependent on the pavement type. Flexible and rigid pavement LEFs may be found in 1993 AASHTO Design Guide and computer program DARWin-2.

Determinations of LEF - Canadian Vehicle Weights and Dimensions Study

The Canadian Vehicle Weights and Dimensions Study was carried out to provide a sound technical basis for heavy vehicle classification and to quantify pavement damage caused by heavy vehicles. While a significant amount of research effort was expended at the AASHO Road Test in the early 1960s, the limited number of pavement types, traffic loadings and environment resulted in speculative extrapolation of the AASHO Road Test data. In the Canadian study, fourteen test sites were instrumented to measure pavement strain and deflection under various configurations (axle loads, axle group, and axle spacings) of a heavy test vehicle. The test sites were located in the five major geographic regions of Canada and the test was carried out using single, tandem, and tridem axles at a range of loads to determine the potential pavement damage in terms of LEF.

The overall average LEFs and the regression equations for the relationships for the Canadian study are shown in Figure 3.2.1.

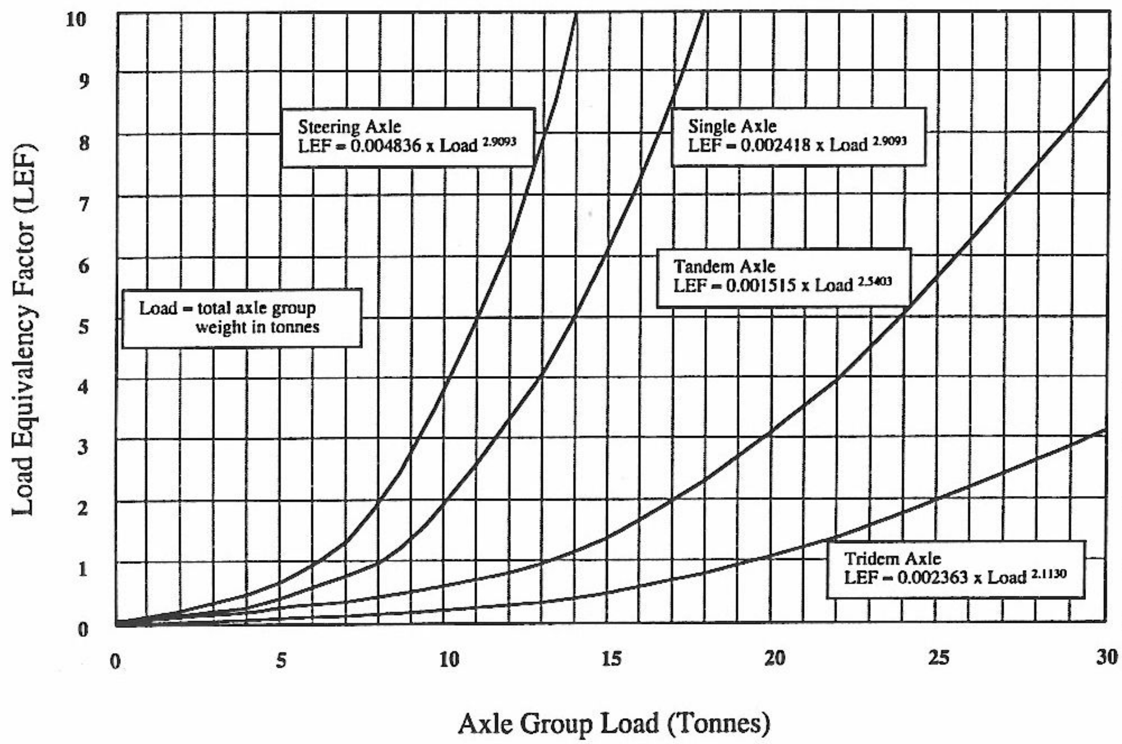


Figure 3.2.1 Average LEF from the Canadian Vehicle Weight and Dimension Study

Measurements of Axle Loads Using Weight-In-Motion (WIM)

The most advanced method of determining the number of ESALs carried by a pavement is to measure the axle loads of each vehicle by using equipment such as Weight-In-Motion (WIM) devices. Weigh-In-Motion is a technology for determining the weight of a commercial vehicle without requiring it to stop on a scale. The system uses Automated Vehicle Identification (AVI) to identify the vehicles; employs technologies that measure the dynamic tire forces of the moving vehicle, and then estimates the corresponding tire loads for a static vehicle. The system may be used in conjunction with an Automated Vehicle Classification (AVC) system. The WIM devices are installed into the pavement and are linked to data acquisition and collection instruments. The data can be sent online or in packages. Several WIM scales have been installed in Ontario mainly where high traffic volumes occur. In 2007, Transport Canada installed four WIM stations in Ontario:

- Hwy 401 (EB & WB, all lanes) Putnam Scales, approximately 2 km west of the WB truck inspection station
- Hwy 401 (EB & WB, all lanes) Gananoque Scales, approximately 1.2km east of the WB truck inspection station (8.5km west of Hwy 37)
- QEW WB Only lanes, approximately 1.9 km west of Ontario St. IC-64
- QEW EB Only lanes, approximately 1.3km west of Fruitland Rd

WIM records axle loads on each axle group, such as single, tandems and tridems in addition to axle spacing. This data is used to determine whether vehicles are within the weight regulations.

WIM also allows the collection of axle load spectra data to facilitate the use of Mechanistic Empirical Pavement Design Guide (MEPDG) and corresponding software application known as AASHTOWare Pavement ME Design, which will be discussed in a later section. Although the WIM can provide axle load spectrum information that is essential for AASHTOWare Pavement ME Design, it requires dedicated commitment and effort to make proper use of this data.

Calculation of ESALs - SHRP Simplified Method

WIM equipment is generally restricted to high traffic volume pavements or research test sections because it is costly. Therefore, traditional estimation methods are employed to determine the design ESAL values from traffic forecasts. The Strategic Highway Research Program (SHRP) has published a simplified method of determining the annual ESAL counts based on the AADT, percentage of heavy vehicles and heavy vehicle factor.

For one lane: $ESAL = 182.5 \times AADT \times TP \times TF$

For two lanes: $ESAL = 182.5 \times AADT \times TP \times TF \times [1.57-0.083 \times \log_e (AADT/2)]$

For > two lanes: $ESAL = 182.5 \times AADT \times TP \times TF \times [1.44-0.083 \times \log_e (AADT/2)]$

Notes: Lanes = Number of lanes for a given direction

Where:

ESAL = Equivalent Single Axle Loads in design lane per year

AADT = Average Annual Daily Traffic (all lanes, both directions)

TP = Percent (divided by 100) of heavy vehicles and combinations

TF = Truck Factor (0.76 for flexible pavement; 1.15 for rigid pavements)

The Truck Factor (TF) is the average ESAL per truck determined from LEF. Truck factor varies with region and with the pavement type. Calibration of TF values to local conditions should be carried out if at all possible, and generalized equations above should be used with caution.

Modified Asphalt Institute Method

A modified Asphalt Institute method has been developed to utilize the truck factor approach while simplifying the input procedure for pavement design engineers. In this method, the following general equation is used:

ESAL = AADT x HVP x HVDF x NALV x TDY

Where:

ESAL = Equivalent Single Axle Loads per Lane per Year

AADT = Average Annual Daily Traffic (all lanes, both directions)

HVP = Heavy Vehicle Percentage (divided by 100)

HVDF = Heavy Vehicle Distribution Factor (% of heavy vehicle in the design lane)

NALV = Number of equivalent axle load per heavy vehicle (Truck Factor)

TDY = Traffic Days per Year

Most pavements are constructed with the same pavement structure across all lanes. The lane subjected to the greatest heavy vehicle traffic is selected as the design lane.

Ontario Method to Calculate ESAL

The following formula is used by Ontario to estimate the number of ESALs. More information can be found in MTO MI-183 Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions [10].

$$ESALs_{Total} = \sum_{i=1}^{i=y} AADT_i \bullet CV\%_i \bullet DF \bullet LDF \bullet CTF \bullet 365$$

where:

$ESALs_{Total}$	= Total or cumulative observed ESALs during the life span of the pavement
y	= Total number of years during the pavement life span
$AADT_i$	= Annual Average Daily Traffic volume for year i, defined as the average 24-hr volume of all highway vehicles (cars, trucks and buses), in both directions, between January 1 and December 31
$CV\%_i$	= Percentage of commercial vehicles (trucks and buses) in the $AADT_i$
DF	= Directional factor, AADT volumes are represent two-way traffic volumes. DF = 0.5 for all sections
LDF	= Lane distribution factor used to assign traffic to the design lane for multilane highways
CTF	= Combined truck factor, defined as the number of ESALs per average truck
365	= Constant to convert daily traffic to yearly traffic

Based on the traffic information from 1995 CVS, a calibration was carried out for Ontario to develop the appropriate truck factor values. The results are summarized in Table 3.2.2 and 3.2.3 below:

Table 3.2.2 Typical Truck Factors for Major Truck Categories

Vehicle Class No.	Descriptions	Truck Factor MTO Regions 1995 CVS				
		East	Central	West	Northeast	Northwest
1	2- and 3-axle trucks	0.35	0.47	0.41	0.56	0.97
2	4 axle trucks	2.77	2.04	1.90	1.76	4.76
3	5 axle trucks	1.49	1.26	1.27	1.40	2.00
4	6 and more axle trucks	4.58	4.33	3.62	3.92	4.34

Table 3.2.3 Typical Truck Factors for Major Truck Classes in Ontario

Major Truck Class	Typical Truck Factor (TF)	Range of Typical TF
2 and 3-axle trucks	0.5	0.05 – 1.00
4-axle trucks	2.30	0.2 – 4.0
5-axle trucks	1.60	0.3 – 3.5
6 and more axle trucks	5.50	2.0 – 7.0

Example of Design ESAL Calculation (Modified Asphalt Institute Method)

- Construction of a four-lane road
- Loaded truck (five axle): 5,000 kg on steering axle; 15,000 kg on both tandem axle
- Travelling 7 days a week all year round
- Design life of 10 years
- Two way AADT = 10,000 with 10% truck traffic (90% truck travel on design lane)
- What is the total design ESALs for this pavement?

The number of ESALs per truck is calculated as follows (refer to Figure 3.2.1):

Steering axle 5,000 kg	=	0.7 ESALs
Tandem axle 15,000 kg	=	1.5 ESALs
Tandem axle 15,000 kg	=	1.5 ESALs
Total ESALs per truck	=	**3.7**

Trucks per day in design direction	=	10,000 x 10% truck x 50% design direction
	=	500
Trucks per design lane	=	500 x 90% truck in design lane
	=	**450**

Design life of 10 years, number of trucks in design lane:

10 years x 365 days/year x 450 truck in design lane per day = **1,642,500**

Design Number of ESALs = 1,642,500 x 3.7 truck factor = **6,077,250**

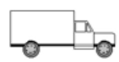
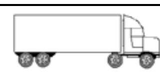
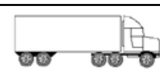
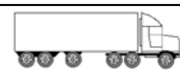
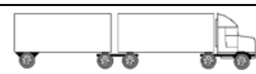
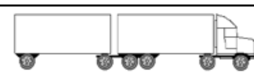
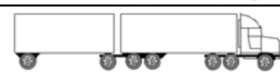
Axle Load Spectrum

Axle load spectrum is another way of evaluating pavement loading during its service life. This method is more compatible with the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG), namely AASHTOWare Pavement ME Design, which will be discussed in later sections.

Axle load spectrum is the distribution of axle weights by axle type. Rather than reducing traffic loading to one 'equivalent' number, traffic loads are determined for each truck class and axle type. Four axle types are included in the analysis: single, tandem, tridem and quadruple axles.

For MEPDG, traffic volumes and axle load spectra are reported separately for ten FHWA vehicle classes. Using Ontario truck classification, the ten vehicle classes are defined the table below, which is taken from the Life Cycle Costing Analysis 2006 Final Report [11].

Table 3.2.4 FHWA Commercial Vehicle Classes

FHWA Vehicle Class	Typical Configuration	Description
4		Two or Three Axle Buses
5		Two-Axle, Six-Tire, Single Unit Trucks
6		Three-Axle Single Unit Trucks
7		Four or More Axle Single Unit Trucks
8		Four or Less Axle Single Trailer Trucks
9		Five-Axle Single Trailer Trucks
10		Six or More Axle Single Trailer Trucks
11		Five or Less Axle Multi-Trailer Trucks
12		Six-Axle Multi-Trailer Trucks
13		Seven or More Axle Multi-Trailer Trucks

The axle load spectra data can be obtained from WIM data or from Commercial Vehicle Surveys (CVS).

Following is an example of the current axle load spectra. This set of axle load spectra is not location specific, but general axle load distribution data representative of southern Ontario. For location specific traffic data, it can be downloaded from the web-based mapping program – *iCorridor* with the following link:

http://www.mto.gov.on.ca/iCorridor/map.shtml

Table 3.2.5 Axle Load Spectra for Single Axle (Southern Ontario)

Axle Weight, kg		Axle load distribution as percentage per truck class									
MIN	**MAX**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	**1,360**	1.80	0.07	0.19	0.28	0.42	0.04	0.39	0.10	0.02	0.44
1,361	**1,814**	0.96	0.33	0.14	0.08	0.42	0.10	0.17	0.09	1.10	0.63
1,815	**2,267**	2.91	5.40	0.89	0.45	2.13	0.62	0.44	0.57	0.02	0.85
2,268	**2,721**	3.99	7.52	0.73	0.70	2.43	0.43	0.89	1.69	3.22	1.21
2,722	**3,175**	6.80	6.65	0.95	0.87	3.55	0.44	0.93	6.75	8.16	1.14
3,176	**3,628**	12.00	11.32	2.12	0.96	7.82	0.62	1.44	5.58	8.73	1.02
3,629	**4,082**	11.70	13.98	4.73	1.51	7.20	1.22	1.48	4.29	8.70	0.99
4,083	**4,535**	11.40	13.94	13.96	3.14	19.16	10.40	4.39	11.03	14.49	4.93
4,536	**4,989**	10.30	10.71	18.40	5.10	13.03	22.56	12.86	14.92	15.75	12.59
4,990	**5,443**	9.00	10.46	24.84	8.07	11.20	40.89	28.90	11.09	15.01	33.61
5,444	**5,896**	7.40	5.04	10.66	3.70	3.96	14.54	15.17	7.09	6.42	17.86
5,897	**6,350**	5.70	4.36	8.60	9.64	6.09	3.05	6.91	10.44	5.54	8.99
6,351	**6,803**	4.30	2.28	4.54	11.08	5.70	1.04	3.37	7.90	4.18	3.33
6,804	**7,257**	3.20	1.95	3.67	13.64	3.76	0.92	3.46	6.14	2.13	2.35
7,258	**7,711**	2.58	1.65	1.45	11.34	2.12	0.90	3.14	3.66	1.42	1.29
7,712	**8,164**	1.80	1.25	1.54	6.99	3.03	0.83	3.46	2.95	1.03	1.58
8,165	**8,618**	1.40	0.80	1.37	5.97	1.45	0.49	2.87	1.75	0.32	1.08
8,619	**9,071**	1.00	0.73	0.42	3.87	1.57	0.28	3.12	0.87	0.83	2.32
9,072	**9,525**	0.75	0.50	0.36	5.90	1.41	0.16	1.96	0.66	0.00	0.72
9,526	**9,979**	0.50	0.51	0.23	2.27	0.95	0.13	1.55	0.38	0.10	0.98
9,980	**10,432**	0.25	0.27	0.04	1.73	0.59	0.11	1.15	0.14	0.08	0.49
10,433	**10,886**	0.15	0.08	0.04	0.23	0.26	0.06	0.38	0.43	0.11	0.21
10,887	**11,339**	0.10	0.06	0.02	0.25	0.18	0.03	0.35	0.19	0.19	0.18
11,340	**11,793**	0.00	0.07	0.04	0.47	0.31	0.03	0.23	0.00	0.71	0.08
11,794	**12,246**	0.00	0.02	0.00	0.04	0.12	0.01	0.11	0.75	1.27	0.17
12,247	**12,700**	0.00	0.01	0.00	0.18	0.11	0.01	0.10	0.00	0.00	0.06
12,701	**13,154**	0.00	0.01	0.00	0.11	0.06	0.01	0.13	0.18	0.24	0.18
13,155	**13,607**	0.00	0.01	0.00	0.00	0.32	0.00	0.10	0.07	0.00	0.00
13,608	**14,061**	0.00	0.01	0.05	0.06	0.11	0.01	0.05	0.18	0.00	0.09
14,062	**14,515**	0.00	0.01	0.00	0.22	0.12	0.01	0.13	0.00	0.00	0.24
14,516	**14,968**	0.00	0.00	0.00	0.13	0.05	0.01	0.10	0.04	0.00	0.10
14,969	**15,422**	0.00	0.00	0.00	0.02	0.14	0.01	0.04	0.03	0.00	0.00
15,423	**15,875**	0.00	0.00	0.00	0.23	0.13	0.01	0.07	0.00	0.00	0.10
15,876	**16,329**	0.00	0.00	0.00	0.09	0.08	0.02	0.04	0.04	0.00	0.00
16,330	**16,782**	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.12
16,783	**17,236**	0.00	0.00	0.00	0.37	0.02	0.00	0.03	0.00	0.00	0.01
17,237	**17,690**	0.00	0.00	0.02	0.06	0.00	0.00	0.01	0.00	0.00	0.04
17,691	**18,143**	0.01	0.00	0.00	0.16	0.00	0.01	0.04	0.00	0.23	0.00
18,144	**20,412**	0.00	0.00	0.00	0.09	0.00	0.00	0.03	0.00	0.00	0.02
Total		100	100	100	100	100	100	100	100	100	100

Table 3.2.6 Axle Load Spectra for Tandem Axle (Southern Ontario)

Axle Weight, kg		Axle load distribution as percentage per truck class									
MIN	MAX	4	5	6	7	8	9	10	11	12	13
0	2,721	5.28	0.00	1.47	0.73	4.02	0.24	0.35	0.00	0.24	0.54
2,722	3,628	10.00	0.00	4.13	0.75	3.89	0.52	0.87	7.65	1.17	3.19
3,629	4,535	11.90	0.00	23.50	1.24	3.99	2.43	1.46	10.35	2.59	6.79
4,536	5,442	9.63	0.00	5.98	2.44	16.68	7.60	2.61	11.54	9.53	5.34
5,443	6,350	8.00	0.00	7.90	4.83	16.58	8.85	6.73	6.55	10.47	7.17
6,351	7,257	7.80	0.00	8.95	13.24	16.90	7.84	9.25	5.05	9.39	4.82
7,258	8,164	6.80	0.00	8.92	12.21	10.77	7.95	7.71	9.90	13.51	3.36
8,165	9,071	6.15	0.00	8.53	9.02	10.58	8.24	5.65	9.52	11.91	2.92
9,072	9,979	5.80	0.00	5.77	4.01	6.35	7.45	4.62	13.19	13.83	2.51
9,980	10,885	5.30	0.00	5.74	7.10	3.29	6.63	3.67	8.52	6.91	2.11
10,886	11,793	4.70	0.00	4.03	6.90	1.63	5.87	3.41	0.00	4.29	2.30
11,794	12,700	4.10	0.00	2.99	3.49	1.48	5.60	3.99	4.20	6.09	3.06
12,701	13,607	3.33	0.00	2.95	2.48	1.17	5.79	5.04	4.57	2.19	2.97
13,608	14,514	3.91	0.00	1.76	2.11	0.60	7.31	5.70	1.76	1.72	4.46
14,515	15,422	2.22	0.00	1.65	3.53	0.66	8.91	7.03	1.58	1.33	6.63
15,423	16,329	1.84	0.00	1.98	1.82	0.89	5.61	8.50	3.49	1.02	10.12
16,330	17,236	1.44	0.00	0.54	2.12	0.35	1.71	7.60	0.00	0.38	10.96
17,237	18,143	0.90	0.00	0.77	5.29	0.10	0.77	6.04	0.00	1.33	9.82
18,144	19,051	0.50	0.00	0.51	4.89	0.00	0.31	4.56	1.44	1.63	5.24
19,052	19,957	0.30	0.00	0.52	3.64	0.07	0.15	2.11	0.00	0.43	1.87
19,958	20,865	0.10	0.00	0.52	3.53	0.00	0.09	1.12	0.69	0.00	1.35
20,866	21,772	0.00	0.00	0.42	1.47	0.00	0.05	0.73	0.00	0.00	0.61
21,773	22,679	0.00	0.00	0.27	1.44	0.00	0.04	0.30	0.00	0.00	0.43
22,680	23,587	0.00	0.00	0.09	0.34	0.00	0.01	0.21	0.00	0.00	0.41
23,588	24,493	0.00	0.00	0.01	0.12	0.00	0.01	0.11	0.00	0.00	0.43
24,494	25,401	0.00	0.00	0.00	0.37	0.00	0.01	0.20	0.00	0.00	0.29
25,402	26,308	0.00	0.00	0.03	0.27	0.00	0.01	0.14	0.00	0.00	0.04
26,309	27,215	0.00	0.00	0.00	0.08	0.00	0.00	0.09	0.00	0.04	0.02
27,216	28,122	0.00	0.00	0.00	0.31	0.00	0.00	0.03	0.00	0.00	0.05
28,123	29,029	0.00	0.00	0.03	0.00	0.00	0.00	0.09	0.00	0.00	0.00
29,030	29,937	0.00	0.00	0.00	0.16	0.00	0.00	0.01	0.00	0.00	0.00
29,938	30,844	0.00	0.00	0.04	0.00	0.00	0.00	0.01	0.00	0.00	0.01
30,845	31,751	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.01
31,752	32,659	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00
32,659	33,566	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.03
33,567	34,473	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34,474	35,380	0.00	0.00	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.01
35,381	36,287	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.03
36,288	38,556	0.00	0.00	0.00	0.04	0.00	0.00	0.01	0.00	0.00	0.10
Total		100	0.00	100	100	100	100	100	100	100	100

Table 3.2.7 Axle Load Spectra for Triple Axles (Southern Ontario)

Axle Weight, kg		Axle load distribution as percentage per truck class									
MIN	MAX	4	5	6	7	8	9	10	11	12	13
0	5,443	0.00	0.00	0.00	4.26	0.00	39.94	4.98	0.00	0.00	6.50
5,444	6,803	0.00	0.00	0.00	9.29	0.00	7.55	9.65	0.00	0.00	11.02
6,804	8,164	0.00	0.00	0.00	10.96	0.00	19.96	9.53	0.00	0.00	6.55
8,165	9,525	0.00	0.00	0.00	0.30	0.00	5.90	7.21	0.00	0.00	3.69
9,526	10,886	0.00	0.00	0.00	14.23	0.00	0.67	5.21	0.00	0.00	2.44
10,887	12,246	0.00	0.00	0.00	1.97	0.00	5.34	5.07	0.00	0.00	2.29
12,247	13,607	0.00	0.00	0.00	4.54	0.00	2.18	4.39	0.00	0.00	2.18
13,608	14,968	0.00	0.00	0.00	2.12	0.00	8.20	4.32	0.00	0.00	4.16
14,969	16,329	0.00	0.00	0.00	12.24	0.00	3.58	4.56	0.00	0.00	4.46
16,330	17,690	0.00	0.00	0.00	0.64	0.00	1.74	4.82	0.00	0.00	4.54
17,691	19,050	0.00	0.00	0.00	0.00	0.00	3.42	5.87	0.00	0.00	3.90
19,051	20,411	0.00	0.00	0.00	0.50	0.00	1.23	5.44	0.00	0.00	7.33
20,412	21,772	0.00	0.00	0.00	0.00	0.00	0.00	6.96	0.00	0.00	11.94
21,773	23,133	0.00	0.00	0.00	9.88	0.00	0.00	6.31	0.00	0.00	14.87
23,134	24,494	0.00	0.00	0.00	3.00	0.00	0.29	5.68	0.00	0.00	8.24
24,495	25,854	0.00	0.00	0.00	6.69	0.00	0.00	4.50	0.00	0.00	3.49
25,855	27,215	0.00	0.00	0.00	9.24	0.00	0.00	2.20	0.00	0.00	1.43
27,216	28,576	0.00	0.00	0.00	4.56	0.00	0.00	1.25	0.00	0.00	0.34
28,577	29,937	0.00	0.00	0.00	5.58	0.00	0.00	0.60	0.00	0.00	0.35
29,938	31,298	0.00	0.00	0.00	0.00	0.00	0.00	0.32	0.00	0.00	0.16
31,299	32,658	0.00	0.00	0.00	0.00	0.00	0.00	0.31	0.00	0.00	0.04
32,659	34,019	0.00	0.00	0.00	0.00	0.00	0.00	0.25	0.00	0.00	0.01
34,020	35,380	0.00	0.00	0.00	0.00	0.00	0.00	0.28	0.00	0.00	0.06
35,381	36,741	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.00	0.00	0.00
36,742	38,102	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00
38,103	39,462	0.00	0.00	0.00	0.00	0.00	0.00	0.05	0.00	0.00	0.00
39,463	40,823	0.00	0.00	0.00	0.00	0.00	0.00	0.09	0.00	0.00	0.01
40,824	42,184	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
42,185	43,545	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
43,546	44,906	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	47,628	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		0.00	0.00	0.00	100	0.00	100	100	0.00	0.00	100

Table 3.2.8 Axle Load Spectra for Quadruples Axles (Southern Ontario)

Axle Weight, kg		Axle load distribution as percentage per truck class									
MIN	**MAX**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**
0	**5,443**	0.00	0.00	0.00	0.00	0.00	0.00	1.25	0.00	0.00	4.32
5,444	**6,803**	0.00	0.00	0.00	0.00	0.00	0.00	4.16	0.00	0.00	8.96
6,804	**8,164**	0.00	0.00	0.00	0.00	0.00	0.00	6.17	0.00	0.00	13.83
8,165	**9,525**	0.00	0.00	0.00	0.00	0.00	0.00	6.06	0.00	0.00	5.35
9,526	**10,886**	0.00	0.00	0.00	0.00	0.00	0.00	4.70	0.00	0.00	0.75
10,887	**12,246**	0.00	0.00	0.00	0.00	0.00	0.00	5.89	0.00	0.00	0.00
12,247	**13,607**	0.00	0.00	0.00	0.00	0.00	0.00	3.56	0.00	0.00	2.19
13,608	**14,968**	0.00	0.00	0.00	0.00	0.00	0.00	2.04	0.00	0.00	2.96
14,969	**16,329**	0.00	0.00	0.00	0.00	0.00	0.00	2.87	0.00	0.00	13.84
16,330	**17,690**	0.00	0.00	0.00	0.00	0.00	0.00	2.37	0.00	0.00	0.82
17,691	**19,050**	0.00	0.00	0.00	0.00	0.00	0.00	3.58	0.00	0.00	3.16
19,051	**20,411**	0.00	0.00	0.00	0.00	0.00	0.00	3.03	0.00	0.00	8.64
20,412	**21,772**	0.00	0.00	0.00	0.00	0.00	0.00	5.41	0.00	0.00	2.03
21,773	**23,133**	0.00	0.00	0.00	0.00	0.00	0.00	6.94	0.00	0.00	5.77
23,134	**24,494**	0.00	0.00	0.00	0.00	0.00	0.00	8.55	0.00	0.00	11.63
24,495	**25,854**	0.00	0.00	0.00	0.00	0.00	0.00	6.94	0.00	0.00	7.89
25,855	**27,215**	0.00	0.00	0.00	0.00	0.00	0.00	4.36	0.00	0.00	0.24
27,216	**28,576**	0.00	0.00	0.00	0.00	0.00	0.00	3.84	0.00	0.00	0.38
28,577	**29,937**	0.00	0.00	0.00	0.00	0.00	0.00	3.72	0.00	0.00	0.00
29,938	**31,298**	0.00	0.00	0.00	0.00	0.00	0.00	3.79	0.00	0.00	0.00
31,299	**32,658**	0.00	0.00	0.00	0.00	0.00	0.00	3.12	0.00	0.00	3.09
32,659	**34,019**	0.00	0.00	0.00	0.00	0.00	0.00	3.61	0.00	0.00	4.15
34,020	**35,380**	0.00	0.00	0.00	0.00	0.00	0.00	1.50	0.00	0.00	0.00
35,381	**36,741**	0.00	0.00	0.00	0.00	0.00	0.00	0.79	0.00	0.00	0.00
36,742	**38,102**	0.00	0.00	0.00	0.00	0.00	0.00	0.35	0.00	0.00	0.00
38,103	**39,462**	0.00	0.00	0.00	0.00	0.00	0.00	1.02	0.00	0.00	0.00
39,463	**40,823**	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.00	0.00
40,824	**42,184**	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.00
42,185	**43,545**	0.00	0.00	0.00	0.00	0.00	0.00	0.16	0.00	0.00	0.00
43,546	**44,906**	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
44,907	**47,628**	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total		0.00	0.00	0.00	0.00	0.00	0.00	100	0.00	0.00	100

Overloads

There are times when it is necessary to move vehicles with gross weights and axle loads that exceed the limitations. The movement of such overloads is controlled by Overweight Permits. The following authorized weights for the transportation of indivisible loads are in place to safeguard pavement integrity:

a) Weight on tires less than 350 mm wide may not exceed 11 kg/mm width as embossed on the tire sidewalls; tires greater than 350 mm wide may not exceed 10 kg/mm width as embossed on the tire sidewall;
b) Weights shall not exceed the Manufacturers Rated Capacity of any vehicle or tire component;
c) Weight limit on a single axle other than a front axle shall be as per Highway Traffic Act (HTA) plus 5,000 kg. Axles equipped with single tires may not exceed 9,000 kgs;
d) Weight limit on tandem axles or two axle groups shall be as per HTA plus 7,000 kg;
e) Weight limit on all other axle units and groups:
   i) with a total spread of less than 3.6 metres shall be as per HTA plus 9,000 kg;
   ii) with a total spread of 3.6 metres or greater shall be as per HTA plus 10,000 kg;
f) Gross weight limit subject to the HTA bridge formula.

During a reduced load period, the weight restrictions are subjected to the lesser of the above weight limits or a maximum of 7,700 kg per axle with single (two) tires and 10,000 kg per axle with dual (four) tires. Overweight privileges are prohibited on all highways designated as being subject to reduced loading and posted as such in accordance with Ontario Regulation 615.

For details of other situations and more comprehensive discussion refer to the website at **http://www.mto.gov.on.ca/english/trucks/oversize/index.html**.

Criteria for Spring (Reduced Load) Restrictions

Spring Load Restriction (SLR) is a factor applied on certain roads and taken into consideration in pavement design and evaluation. As ambient temperatures increase after the winter season, thawing begins at the road surface and continues downwards through the pavement structure. The most critical period for pavement stability is when the asphalt pavement is only partially thawed and the granular base or subgrade is unable to provide adequate support. Loading of the asphalt pavement surface during this time may result in significant strains at the bottom of the surface layer and can lead to premature cracking. Concrete pavements depend less on subgrade support and are normally not subjected to spring load restrictions.

The resilience moduli of the unbound materials and subgrade are usually at their lowest support level when excessively wet. Moduli of some fine grained subgrade soils when wet can be as low as 20% to 30% of their summer or fall values.

Under the Highway Traffic Act, the province enforces reduced load restrictions on trucks to protect Ontario’s highways during spring thaw, when road damage is most likely to occur. The Highway Traffic Act of Ontario provides details for the imposition of reduced load restrictions during the spring thaw, for certain secondary highways. These highways have not been designed to carry full loads during this period of weakened condition and could suffer accelerated damage if full loads were permitted.

The restrictions are applied initially to applicable highways in the southern part of Ontario, and applied later in the north. Removal of the restriction is specified to minimize the reduced load period, but also to allow for sufficient recovery of strength. At present, removal of restrictions is specified on a data basis depending on general regional conditions; however, local concerns are taken into consideration. Currently a study is underway using the temperature data from the Road Weather Information System (RWIS) to control load restrictions on gravel and surface treated highways in Ontario [12]. The objective of the study is to examine the correlation between temperature data and frost formation / thawing to improve the scheduling of load restrictions.

More recently, Tire Pressure Control Systems (TPCS) are being used in certain areas of northern Ontario to permit trucks to carry heavier loads during spring load restriction periods with reduced tire pressure. Lowering tire pressure increases the tire footprint. The increased surface area touching the pavement reduces the impact to pavement. Preliminary results indicate that the use of TPCS may reduce the spring load restriction period by up to 4 weeks.

3.2.4 ENVIRONMENTAL CONDITIONS

The two main factors causing pavement deterioration are traffic loadings and environmental conditions. Environmental factors cause deterioration of pavements predominantly through excess moisture, frost induced differential heave, settlement of subgrade and thermal cracking induced by temperature variation. These factors, described in the following section, must be taken into consideration in pavement design.

Temperatures

The environmental condition most damaging to pavements in Ontario is the severe winter weather [13]. The penetration of frost into the pavement structure and subgrade can cause heaving and growth of ice lenses. Stresses introduced in the pavement cause distortion and cracking which may require repeated maintenance. Stresses may be introduced in the pavement if snow removed from the travelled portion of the highway is piled on the shoulder. This can lead to the development of longitudinal cracks along and adjacent to the centreline of the pavement. The weakening of granular and subgrade caused by an increase in moisture due to spring thaw may result in loss of stability. Frost heaving and transverse cracking of pavements due to low temperatures results in annual losses in riding quality and a reduction in the life of a pavement.

During the summer, high temperatures can cause pavement problems such as rutting due to softening of the asphalt pavement surface and "blow-ups" due to thermal expansion associated with older concrete pavements with long slab lengths. These types of problems can usually be overcome through proper design.

Moisture and Rainfall

Moisture and rainfall can play a significant role in the performance of the pavement structure. It is sometimes difficult to carry out good road construction practices in areas that have a high water table and predominantly cohesive or fine-grained subgrade material. This situation can be aggravated during prolonged periods of heavy rains. These types of wet soils cannot be compacted adequately resulting in the need to use alternate granular type materials at increased costs. In areas of excessive rainfall, special drainage designs may be required.

Snow

Snow drifts and blowing snow create safety hazards and winter maintenance problems. These undesirable conditions can be minimized or eliminated with such treatments as grade line control on fills, the flattening of cut backslopes and removing obstructions such as dense trees and rock cut backslopes to a distance of 15 times the height of the obstruction [2]. In addition, snow windrows on shoulders create differential thawing across the pavement width which results in temporarily saturated base and subbase conditions and associated loss in structural strength.

Icing

Icing, also known as "black ice", is formed on the pavement surface under certain conditions of temperature and humidity. It has been observed on the pavement in shaded rock cuts. At locations insulated with expanded extruded polystyrene, icing, which is caused by the trapping of ground heat, can largely be overcome with the appropriate design of the insulation below the pavement structure. Refer to Ministry Directive C-17 [14] for more details.

3.2.5 FROST PROTECTION

Causes and Effects of Frost Action

The two principal damaging effects of frost action in highways are: (1) differential frost heaving, and (2) reduction of subgrade support strength during the thawing period. Frost heaving is caused by formation of ice and ice lenses within the soil. For ice lenses to form, three conditions must be present:

a) The soil is frost-susceptible;
b) The soil temperature must be below freezing; and
c) Sufficient water must be present at or near the freezing temperature.

The removal or modification of any one of these conditions will usually restrict frost heaving. Differential frost heaving creates a hazard for the driving public and it contributes to the deterioration of the overall pavement structure. More information on frost action on pavement can be found in Reference [15].

The reduction of subgrade support strength during the thawing period may be due not only to a change in the moisture content or density of the material but also to the form in which moisture is present or to the degree of saturation of the soil. Fine-grained soils are more likely to be affected by frost action than granular soils.

Frost-Susceptibility of Soils

Fine-grained soils that have high capillarity and low cohesion are especially subject to frost heaving. These soils can be identified and classified by laboratory testing. The boundaries established for susceptibility to frost heave are related to the percentage of very fine sand and silt in a soil [16a]. The frost susceptibility of a soil is rated as follows:

Grain Size (5 to 75 μm)	Susceptibility to Frost Heaving
0 to 40%	Low
40 to 55%	Moderate
55 to 100%	High

Other factors that affect the degree of frost susceptibility are as follows:

- Rate of heat removal
- Temperature gradient
- Mobility of water (e.g., permeability of soil)
- Depth of water table
- Soil type and condition (e.g., density, texture, structure, etc.)

Frost Depth

An important consideration in pavement design and in the treatment of differential frost heave is the depth of frost penetration. Due to the wide range of temperatures encountered across the province, the depth of frost penetration varies considerably. As a rule, frost penetration is deeper in dry, well drained soils than in saturated soils.

An estimate of the frost penetration depth anywhere in Ontario can be made by referring to frost penetration depth contour lines for southern and northern Ontario, Figures 3.2.2 and 3.2.3, taken from Reference [17]. These depths refer to the depth of frost penetration (in metres) beneath the paved highways under plowed conditions.

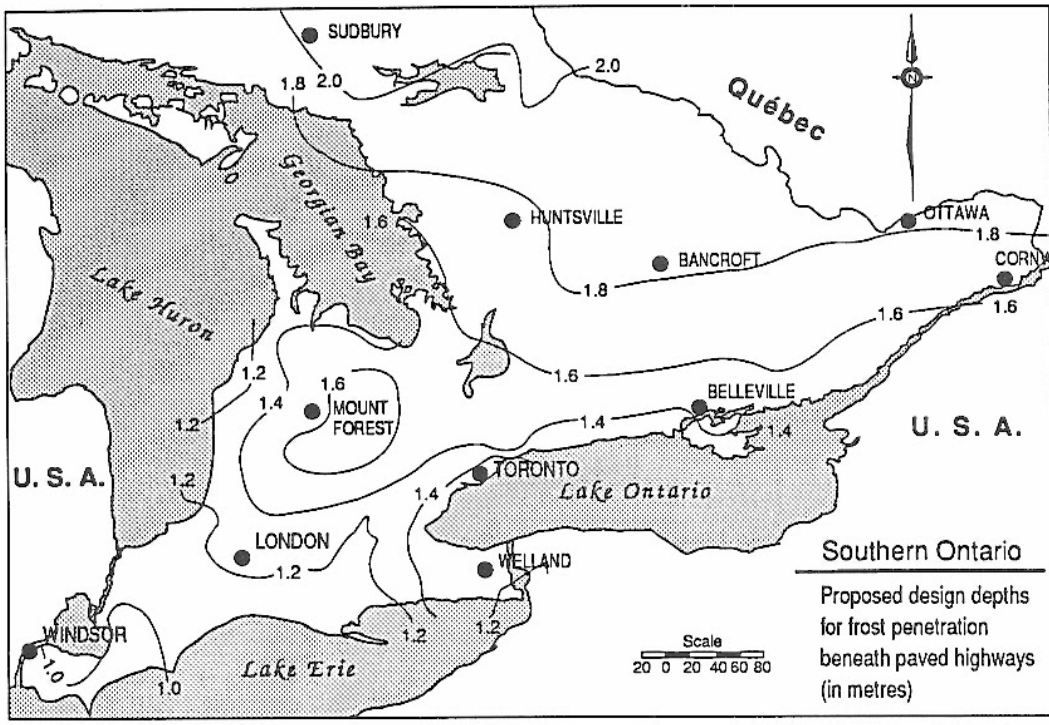


Figure 3.2.2 Frost Penetration Depths of Southern Ontario

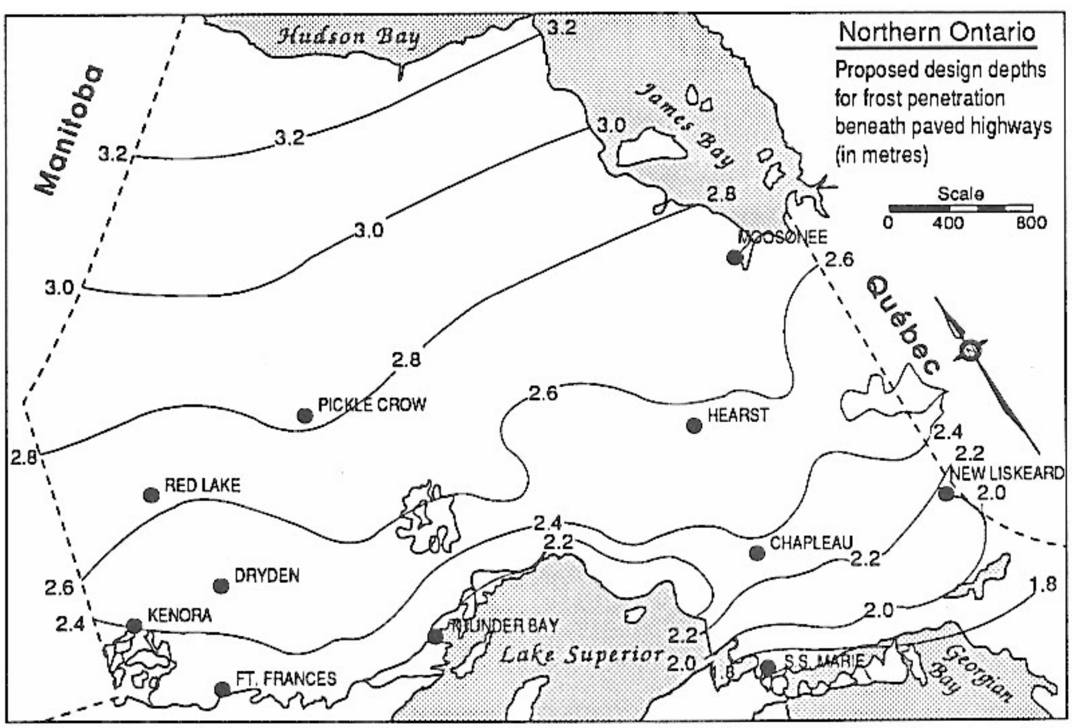


Figure 3.2.3 Frost Penetration Depths of Northern Ontario

Frost penetration depths can also be determined by knowing the freezing index from a design curve developed by MTO, Figure 3.2.4. The freezing index is described as the "cumulative number of degree-days" versus "time" for one freezing season. The freezing index information can be obtained from freezing index contour maps published by the National Research Council of Canada and reproduced in MTO Soils Manual [18].

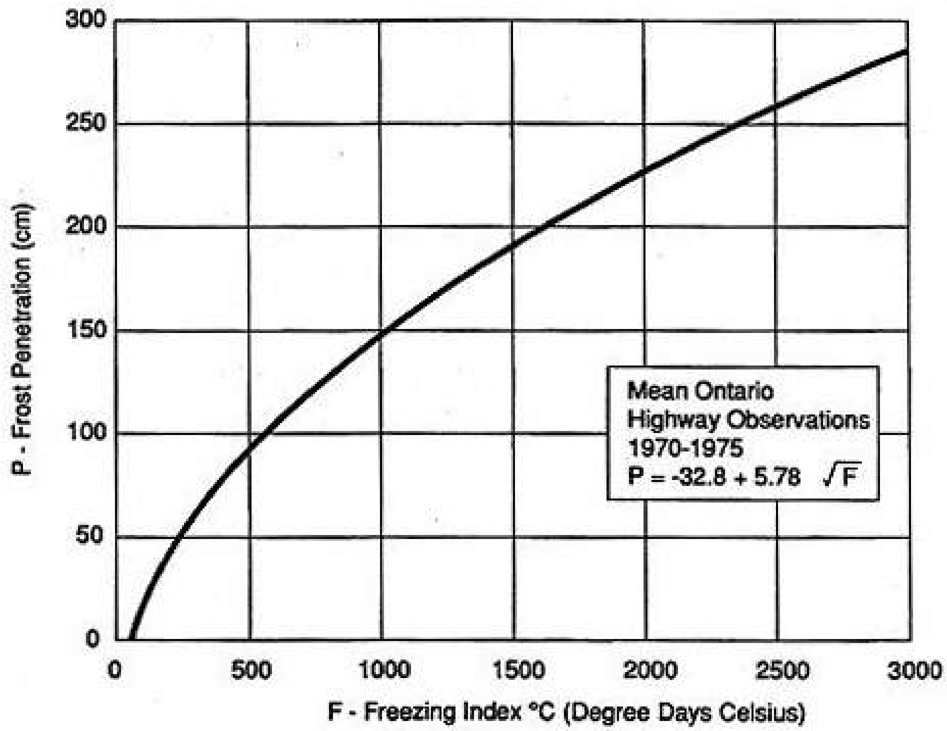


Figure 3.2.4 Design Curve of Observed Frost Penetration

Methods of Limiting Frost Action and Differential Frost Heaving in Pavement Designs

The depth of frost penetration makes it impractical to provide pavement thickness that would prohibit frost penetration into the subgrade. Such thicknesses would far exceed the requirements for bearing capacity; thus, in practical terms, construction would be uneconomical. The following should be considered in design to minimize the effects of frost:

a. Utilize uniform subgrade soils with respect to their susceptibility to frost heaving. This is accomplished by determining the type of subgrade material by laboratory grain size analysis and its susceptibility to frost heaving as described above.

b. Design the pavement structure based on the reduced strength of the subgrade soil during the critical spring thaw period. This provides a pavement which is adequate for the design loads during the thawing period but which during other periods will have a load carrying capacity in excess of that required.

c. Intercept the water before it enters into the frost-susceptible areas of the roadway through the provision of adequate side ditching and/or subdrains.

d. Prevent surface water from entering the granular bases and subgrades with paved shoulders and/or edge subdrains.

e. Ensure appropriate treatment of transition points and boulders in the subgrade [16b]. These are covered by Standards OPSD-205.01 to OPSD-205.05 and OPSD-204.01. The appropriate treatment of culverts and sewers is covered by OPSD-803.030 to OPSD-803.031 [16c].

f. Use non frost-susceptible granular backfill material to avoid differential heaving between the pavement approaches and the structures in accordance with OPSD-3289.01 to OPSD-3289.03 (formerly DD350 to DD356).

Methods of Treating Frost Heaves

Since soils of uniform frost-susceptibility are used in the construction of the subgrade, differential frost heaving is not a serious problem in the performance of our pavement structures. However, on occasion and in particular for Northern Ontario, differential frost heaving does occur which will warrant treatment from a safety point of view [16a], [16b].

Excavation -The principal method of treating these differential frost heaves is the removal of the frost-susceptible material by excavation (OPSD-205.06). The design value for the depth of excavation varies with the depth of frost penetration and is selected so as to remove as much of the frost-susceptible materials as possible bearing in mind the amount of excavation involved and the fact that it must be properly drained. Majority of pavements in Ontario will heave, so the purpose of this treatment is to achieve uniformity in heaving and pavement performance.

Drainage - Differential frost heaves of low severity can often be treated successfully by improving ditching and/or ditch cleaning or the installation of perforated subdrains beneath the shoulder of the pavement. The depth of the ditch or drain is dependent on the availability of an outlet to remove the collected water.

Expanded Extruded Polystyrene - A method of controlling the effects of differential frost heaving is by reducing the depth of frost penetration in the soil by means of insulating materials. Expanded extruded polystyrene has been found suitable for this purpose. Through earlier experiences with this treatment of differential frost heaves, it was found that while the main heave was eliminated, bumps developed at either end of the insulated area. To minimize the bumps, it was found that controlled heaving with suitable end treatments would be acceptable (OPSD-514.01 and 514.02). This technique allowed the frost to penetrate below the insulation. Differential heaving is reduced to the extent where any hazard to the motorist is eliminated and driving discomfort is minimal [19].

Pavement icing has been perceived to be a problem where the polystyrene has been used for insulation purposes. In the interest of safety, frost heave treatments with expanded extruded polystyrene insulation should be avoided on curves, crests of hills, at intersections, railroad crossings and at any other location where a demand for increased traction or braking is foreseen.

The policy indicates that the use of polystyrene in potentially hazardous areas should be avoided. The policy regarding frost heave treatments using expanded extruded polystyrene insulation is covered by Directive C-17 [14]. Essentially, it indicates the use of polystyrene with maximum thickness from 25 mm to 40 mm depending on location.

Construction traffic must not be allowed to run on the expanded polystyrene until a sufficient depth of granular base has been laid over it for protection. The compacted granular cover should be no less than 300 mm in depth, and it should be increased at end of pavement structure.

Polystyrene has other applications in highway design and construction for insulation purposes. A very important application is at weigh scales where vertical movement between the concrete approach slab and the scale platform must be kept to a minimum to ensure the accuracy of the scale. Greater thicknesses of expanded polystyrene are used at these truck inspection stations (OPSD-511.01 and OPSD-511.03).

Polystyrene is also considered for use on new construction from a practical point of view and when there is no other economic alternative for the treatment of potential frost heave areas.

3.2.6 PAVEMENT DESIGN REQUIREMENTS

Serviceability

Pavements are designed and constructed to provide a safe and comfortable ride at normal speeds for all vehicles in all types of weather. The life span of a pavement is generally the period of time, in years, between the time the highway is opened to traffic and the time the pavement reaches its terminal serviceability level. For pavements, the ability to service the traffic that the pavement was designed for is a measure of serviceability.

Immediately after construction, new pavements generally have very good riding surfaces. The quality of the ride provided by a pavement is reduced over time due to various factors such as traffic, age, and environmental conditions to a point where the riding surface is no longer acceptable and rehabilitation is required. The need, extent, and timing of the rehabilitation, depend upon the serviceability that a particular road is expected to provide. The serviceability can be determined, in part, by pavement evaluation or pavement condition rating which rates two different physical parameters: (1) the riding quality of the pavement surface, and (2) the extent and severity of distress manifestations. The type of distress and the extent and severity of its occurrence are all very useful indicators of structural adequacy, material deficiency and probable rate of subsequent deterioration [20], [21].

The detailed procedures for determining pavement condition ratings are described in Chapter 5.

Design Life and Service Life

Pavements are designed to last for a specific number of years prior to the first resurfacing, and this is called the design life. The service life of a new, reconstructed, overlaid, or restored pavement is the time from initial construction until the pavement has deteriorated to the point at which significant rehabilitation or reconstruction is needed. Refer to the Life Cycle Costing Analysis (LCCA) study for typical expected service life for each pavement type, reference [22]. Each subsequent resurfacing is expected to last at least 10 years as indicated in Directive B-82 [23].

3.2.7 LIMITATIONS

Many factors can affect the scheduling and total cost of the project. These factors must be considered in the preparation of geotechnical designs. Some of these factors are as follows:

Time Constraints

- The nature of the work and the time available to carry out the work, i.e., emergency or stop-gap treatments, pavement repairs.

- Need for lane or road closings and evening work.
- Type of pavement to expedite construction.

Costs

- The designs must be cost-effective.
- The service life of pavements must be kept in mind. If the design is too elaborate and costly, the project may be dropped from the program. If it is under-designed, it may not provide the service life expected.

Quantity and Availability of Materials

- Maximize the use of local materials to minimize initial construction costs.
- Specify materials that are locally available and/or can be produced when required.
- Recognize the capability and efficiency of contractors' equipment and plants, bearing in mind their limitations in producing the desired materials, i.e., high recycling ratios (greater than 50:50) should not be designed for a job where batch plants exist in the area.

Timing of Contract

- The geotechnical designer has very little control when the work is carried out, therefore, the designs should allow for a wide range of climatic conditions without affecting the quality of work.
- If the timing of the contract or parts of the contract are critical, constraints may be required through staging of the work and special provisions.
- Deferred projects should be reviewed for any updates required.

Contract Bidding and Marketplace Constraints

- Ensure that the designs can be built by contractors who have the equipment, knowledge and trained staff to perform the work.
- The designs should be practical for construction purposes.
- A large local demand for processed materials may increase its cost significantly. Consideration should be given to alternative designs.
- The Estimating Office is in a position to provide cost information for design purposes.

3.2.8 PAVEMENT SELECTION PROCESS

Selection of pavement type is determined during the preliminary design stage of a project that often takes place several years before construction.

A preliminary soil survey is usually carried out to determine the general soil types and granular resources along the route. At this stage, the pavement designer presents a number of alternative designs of equivalent strength for comparison.

Pavement design is carried out by Regional Geotechnical staff or by pre-qualified geotechnical consultants listed in the ministry's Registry, Appraisal and Qualification System (RAQS) under the Pavement Engineering specialty. The geotechnical component of design projects are rated as low, medium and high complexity. Only consultants identified in RAQS under the designated specialties are eligible to carry out pavement design.

Pavement Design Reports, provide recommendations for the most appropriate and cost-effective pavement rehabilitation strategy based on the findings of the soils investigation. Regional Geotechnical staff review pavement design reports for quality and consistency.

In addition to the design factors considered in the pavement selection process, the present value costs including resurfacing of each pavement type alternative after its expected initial life must be included. All this cost information can be obtained by utilizing the Ontario Pavement Analysis of Costs (OPAC 2000) System as described in Section 3.3.1 or Highway Costing (HICO). All flexible pavement structures are to be designed a minimum thickness that will produce a life of 10 to 12 years, as predicted by OPAC 2000.

For rigid pavements, OPAC 2000 or HICO is to be used only for calculating the initial cost. Total life cycle costs for rigid pavements can be computed utilizing this initial cost plus the maintenance costs and salvage values over the design life, as detailed in Section 3.3.

The selection of freeway pavement designs is carried out using the Life Cycle Cost Analysis (LCCA) process. This includes high-volume roadways with greater than 1 million ESALs per year (current or projected within 5 years), all freeways and 400 series highways, and for all concrete pavements (any facility type). LCCA shall compare the life cycle costs of at least one asphalt pavement design and one concrete pavement design. The total present worth (PW) cost of each pavement design is the sum of the cost of initial construction, the PW costs of individual preventive and rehabilitation activities, and the PW salvage benefit. A 50-year analysis period is stipulated with the current discount rate published by the Ministry of Finance. The lowest 50-year life cycle cost option is recommended for the pavement selection. [22]

For any other rehabilitation and expansion projects, LCCA is also required in order to select the most cost effective pavement alternatives, and the analysis period is usually 30 years life cycle.

3.3 STRUCTURAL DESIGN

Introduction

There are many methods of pavement structural design available [24], and designers' opinions, with respect to suitability of design, vary from place to place. It should be mentioned that pavement structural design is not an exact science, and that there are many variables involved that make it complex. The structural design should be based on extensive investigation of specific project conditions and needs.

3.3.1 STRUCTURAL DESIGN METHODS

Structural design of a pavement section is determined by assessing different combinations of pavement surface, base, and subbase layers and selecting the optimum design that meets the specific project requirements.

Different pavement design methodologies have evolved over the years. The following pavement design methods are discussed in this section [1]:

1. Experience-based methods using standard sections.
2. Empirical methods in which relationships between some measured pavement response, usually deflection or field observations of performance and structural thickness are utilized.
3. Theory-based procedures, using calculated stresses, strains or deflections. These are also known as mechanistic-empirical methods.

Experienced-based procedures rely on existing successful pavement designs and use of standard sections. Standard layer thicknesses are adopted for various conditions of traffic, subgrade type, highway classifications, drainage and other factors. The simplicity of this approach lends itself to inappropriate extrapolation of standard designs to areas with differing site conditions and requirements.

Empirical procedures often use the results of measured responses, such as deflections obtained by Benkelman Beam (BB) or Falling Weight Deflectormeter (FWD), on different pavement structures to establish limits for the adequate pavements for various volumes of traffic. Many of the limitations of the experienced-based methods also apply to empirical-based methods.

Theory-based procedures rely on calculated values of stress, strain and deflection at some location in the pavement structure. This information is used to observe performance under various conditions and axle load repetitions over the pavement's life. The theory-based procedure was developed using full-scale road tests and experimental field sections. The results of the Long Term Pavement Performance (LTPP) studies of the Strategic Highway Research Program (SHRP) have contributed substantially to the development of theory-based procedure such as the AASHTO Mechanistic Empirical Pavement Design Guide (MEPDG). Theory-based procedure also relies on some empirical components to make the design consistent and reliable. Local calibration of environmental and climatic variations is also required for theory-based design procedure. In certain more rigorous design situations, excessive structural damage in terms of fatigue cracking, rutting and low-temperature cracking can be predicted.

Pavement design methodologies are discussed in details in the following subsections.

3.3.2 PAVEMENT TYPES

Pavement consists of all structural elements or layers, including the shoulders, above the subgrade. Although subgrade is not part of the pavement structure, its characteristics such as strength or load carrying capacity and drainage are implicitly included.

Pavement may be flexible (asphaltic concrete), rigid (Hydraulic Cement Concrete) or a composite that is a combination of flexible and rigid pavements, such as an asphaltic concrete surface on a Hydraulic (Portland) Cement Concrete (HCC or PCC) base.

Figure 3.3.1 below illustrates the typical cross section of different pavement types.

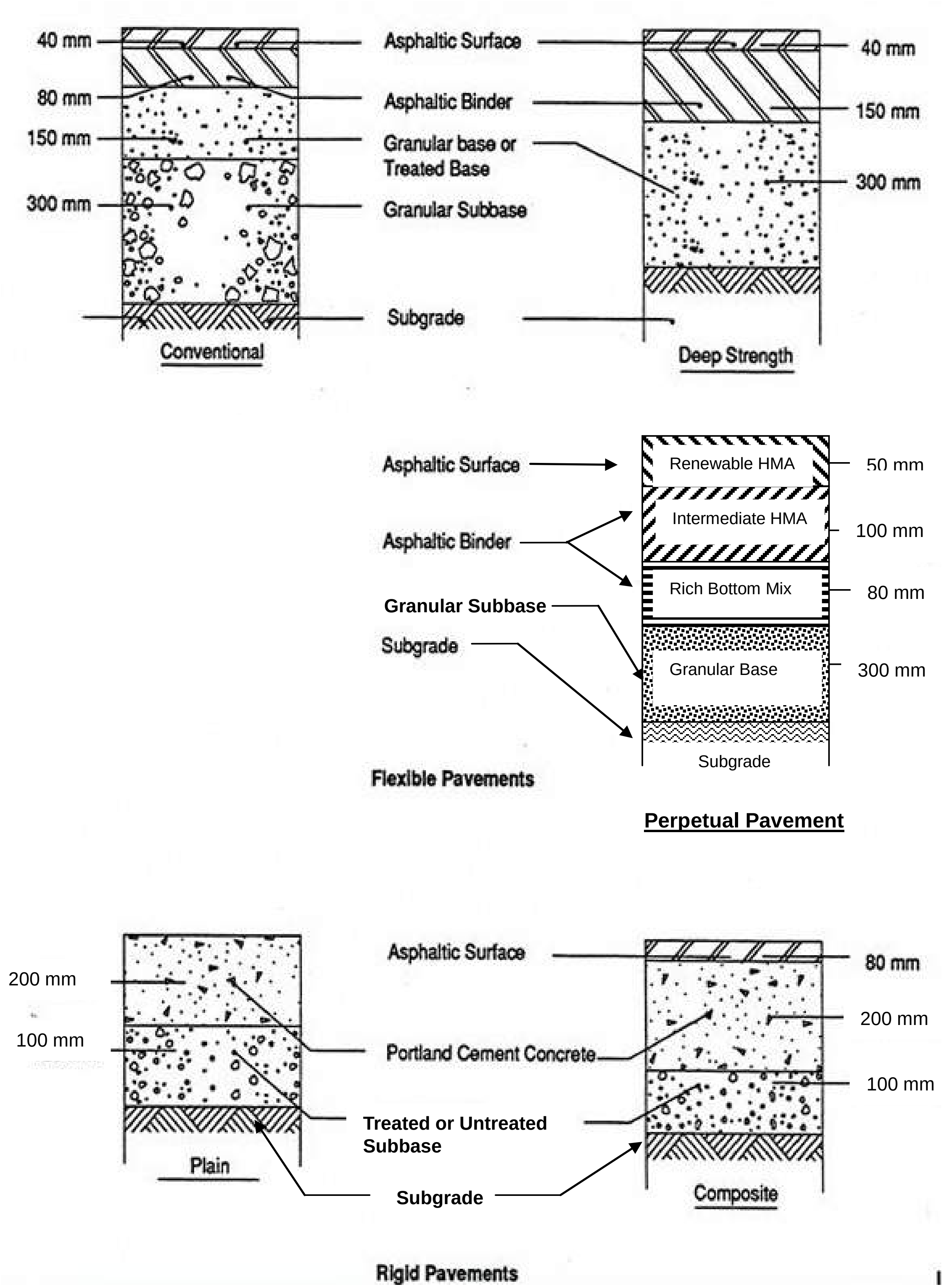


Figure 3.3.3 Pavement Types

Flexible Pavement

Asphaltic concrete pavement also known as hot mix asphalt (HMA) concrete pavement is the most prevalent pavement type in Ontario. Flexible pavements can be separated into a variety of sub-types as indicated below:

- Conventional – less than 150 mm of asphaltic concrete on granular base and subbase
- Deep strength – 150 mm or more of asphaltic concrete on granular base
- Perpetual design – 200 mm or more of asphaltic concrete with or without the rich bottom mix (RBM) layer on granular base
- Semi-rigid – HMA placed over cementitious stabilized materials.

Conventional flexible pavement is the common pavement for most regions in Ontario and, consists of an asphaltic concrete surface layer with underlying layers of asphalt concrete binder or leveling course layer, granular base and granular subbase. The base and subbase layers are sometimes stabilized or bound. Figure 3.3.2 illustrates the typical cross-section of a conventional asphaltic concrete pavement.

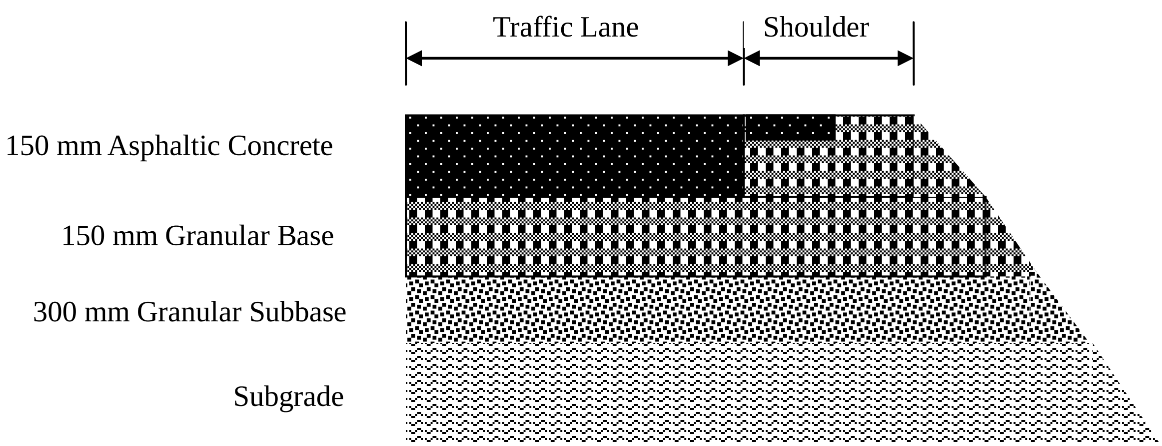


Figure 3.3.2 Typical Cross-section of Flexible Pavement

Pavements having asphaltic concrete thicknesses of 150 mm or more and founded on a minimum 150 mm thick granular base are known as "deep strength". Where the asphaltic concrete layer (150 mm or more) is placed directly on subgrade, the pavement is referred to as a "full depth" design.

Deep strength and full depth designs are used on higher volume roads and/or where local granular subbase materials are scarce. Where total roadbed thickness is reduced in this manner, provision must be made to ensure adequate frost protection. The ministry seldom uses full depth pavement structures as granular material in the base and subbase serve as a drainage layer which is critical to pavement performance.

Perpetual pavement is a long-lasting structural design, construction and maintenance concept projected to perform 50 years or more and requires only periodic renewal of the top surface course. The benefits of using perpetual pavement include ease of maintenance, low future road user impacts, and quick and easy repair.

Perpetual pavement is a thick asphalt pavement consisting of a flexible, fatigue-resistant bottom layer; a strong rut-resistant middle layer; and a smooth, durable renewable surface layer. Two approaches can be used to resist fatigue cracking in the bottom layer. The total pavement thickness may be designed thick enough to render the tensile strain at the bottom of the base layer is insignificant. Alternatively, the HMA bottom layer can be constructed with extra-flexible HMA. This is achieved by increasing the asphalt cement content, to create an asphalt-rich bottom layer called Rich Bottom Mix (RBM). Combinations of the two approaches also work. The intermediate layer is designed to carry most of the traffic load. It must be rut-resistant and durable. Stability can be achieved by using stone-on-stone contact in the coarse aggregate and using asphalt cement with the appropriate high-temperature grading. The top renewable surface course is designed to resist surface-initiated distresses such as top-down cracking and rutting.

On low volume roads, surface treatment consisting of one or more applications of asphaltic material and stone chips is used in lieu of asphaltic concrete.

Rigid Pavement

A rigid pavement generally consists of plain, composite or reinforced hydraulic (Portland) cement concrete slabs placed on a base or subbase, or sometimes directly on subgrade. Hydraulic cement concrete (HCC) has a high modulus of elasticity, which makes it very rigid. The rigidity and flexural strength of a concrete pavement allow it to distribute loads over large areas. This has led to a set of design requirements and procedures, which are described in the following sections. It is important to note that there are many design variables which need to be optimized for a successful design. Thickness is only one such variable; jointing and base support are equally important issues to address.

Composite pavements (concrete slabs with bituminous overlays) are also considered as rigid pavements. Although initial construction costs are typically higher, rigid pavements usually provide greater load carrying capability, a longer service life, and reduced maintenance costs when compared to flexible pavements.

Rigid pavements are those which include Hydraulic Cement Concrete (HCC) within the pavement structure. The four most frequently used rigid pavement types are:

a. JPCP – Jointed Plain Concrete Pavements with either dowelled or undowelled joints.
b. JRCP – Jointed Reinforced Concrete Pavements typically with dowelled joints.
c. CRCP – Continuously Reinforced Concrete Pavements (CRCP), constructed without joints.
d. Prestressed pavement - Post-Tensioned Concrete Pavements (PTCP) or Precast Pre-stressed Concrete Pavement (PPCP).

The majority of concrete pavements built in Ontario are jointed plain concrete pavement.

HCC pavements usually include a granular subbase. Newer HCC pavements also typically include a permeable base layer between the slab and granular subbase. The permeable base layer often consists of a stabilized material, such as asphalt cement treated or cement treated Open Graded Drainage Layer (OGDL). Moreover, HCC pavements can be constructed directly on subgrade. Figure 3.3.3 is a typical cross-section of a rigid pavement structure with widened lane extended into the shoulder.

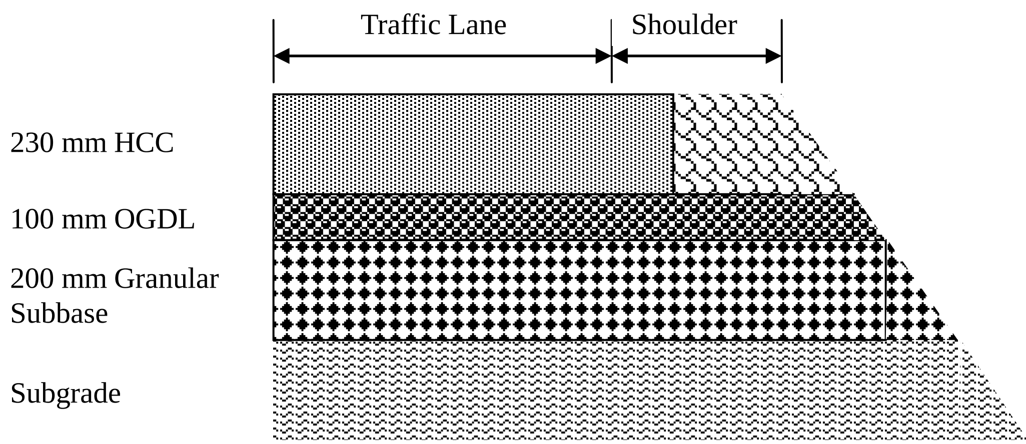


Figure 3.3.3 Typical Cross-section of Rigid Pavement

Design Types

There are several concrete pavement types, but the most common type is the Joint Plain Concrete Pavement (JPCP). Ontario is only adopting the use of JPCP.

The JPCP pavement structure is built without reinforcing steel and may or may not include dowels in the transverse joints. Typically, a maximum joint spacing of 4.5 m is used to minimize transverse slab cracking. The ministry currently specifies the use of JPCP with dowelled joints with random joint spacing (3.5, 4.5, 4.0, 4.3 m) to minimize repetitive noise issues related to concrete pavement.

Reinforced concrete pavements usually contain a steel mesh, designed to hold together the one or two cracks that usually develop between the joints. Joint spacings are typically higher but not more than 13.0 m. Due to the longer slab lengths, there is increased joint opening in cold seasons necessitating the use of dowels at joints to provide load transfer. Generally, reinforced pavements have lost favour in northern climates. JRCP usually has a higher capital cost. MTO has experienced problems with reinforcement corrosion and joint contamination with incompressible material which has lead to “blow-ups”.

Continuously reinforced pavements, as the name implies, contain relatively heavy continuous steel reinforcement in the longitudinal direction to eliminate joints. The steel is designed to ensure that transverse cracking develops at close intervals. The reinforcement also holds the cracks together tightly to maintain a high degree of load transfer. The initial cost for this pavement is significantly higher than for a plain pavement; however, they have been used by various agencies in North America and Europe on high volume facilities, usually in more temperate climatic zones like the Southern US.

Prestressed or post tensioned pavements have been used in specialized locations, predominantly airports, where the tensioned steel can be utilized to reduce the concrete thickness. It is more difficult to take advantage of this in a highway pavement and as a result, usage to date has generally been experimental in nature.

Base, Subbase and Subgrade

Concrete has a modulus of elasticity ranging from 26 to 30 GPa and thus has a high degree of rigidity. Concrete pavements also have substantial flexural strength as indicated by flexural strengths generally averaging above 4.4 MPa. This rigidity and flexural strength enable concrete pavements to distribute applied loads over a large area, which means the pressure exerted on the underlying foundation are a fraction of the applied load. Tests have shown that for a tire with applied load of 5,400 kg the pressure beneath a 200 mm slab is only 20 to 50 MPa and radiates 3.0 m around the load point. A number of studies have substantiated this information.

Concrete pavements do not require bases or subbase for structural support. It is, however, very important to provide uniform support because of “beam action”. Bases are often used on higher volume roads to facilitate drainage and minimize erosion of foundation materials or to expedite construction on lower volume facilities.

Non-uniformity of subgrades may lead to differential movement due to frost heave or expansive soils. In concrete pavements extensive removal or sub-excavation of poor subgrade material is generally less economical than control of the problem through subgrade preparation. For example, by using subgrade stabilization in expansive soils or subdrains to eliminate or reduce subgrade moisture levels, frost heave can be significantly reduced. However, if a very deep frost depth condition exists, the reduction of frost heave may only be slight.

A base material is required for facilities which have frequent passage of heavy axle loads, and are founded on subgrade soil which is susceptible to erosion. Erosion occurs when heavy vehicles pass over transverse joints causing rapid movement of free water which can remove or relocate fine materials. Redistribution of fine material can lead to a difference in elevation at the joints. This mode of failure, often referred to as faulting, was common prior to the extensive use of dowelled joints and non-erodible base materials.

Faulting and erosion parameters have been incorporated into the latest version of the Portland Cement Association (PCA) and AASHTO thickness design procedures. It is, however, important to remember that water and poor base conditions cannot be corrected by increasing concrete thickness.

Today many high volume facilities are constructed on permeable layers which significantly reduce the amount of time water is present below the slab. They also eliminate the influence of erodible fine material. Preliminary findings indicate good performance.

The use of cement treated stabilized bases (CTB) has also been used extensively to provide an erosion resistant base. However, the ministry typically specifies the use of asphalt treated or cement treated open graded drainage layer (OGDL) as the stabilized base.

Joints

Jointing of concrete pavements is required to control the geometry and interval of longitudinal and transverse cracking which would otherwise occur randomly. Joints maintain the long term structural capacity and riding quality of the pavement. Joint design is a very important element in the design of concrete pavements and a well designed system will:

- Control transverse and longitudinal cracking
- Accommodate slab movements
- Provide desired load transfer
- Provide a reservoir for joint sealant
- Divide the pavement into practical construction increments

The development of concrete pavement joint design is based on theoretical studies, laboratory tests, experimental pavements and performance evaluations of existing pavements. Joint design has evolved considerably from a time when no joints were incorporated in concrete pavements to the typical short joint spacing used today. Joints control cracking due to stresses caused by temperature change, drying shrinkage, moisture differentials, thermal gradients, and traffic loading.

Initial cracking of a concrete pavement is due to a combination of factors. The most significant initial cracking is shrinkage due to temperature change and loss of water. As concrete hydrates it generates heat but then begins to cool shortly after final set, causing the pavement to contract. After hardening, the pavement is also influenced by temperature and moisture gradients which cause the pavement to curl and warp.

Recent studies indicate that concrete pavement should be designed with a widened shoulder to accommodate heavy truck traffic on the driving lane thereby eliminating early longitudinal cracking of the concrete pavement. Rather than a 3.75 m wide driving lane, a 0.5 m integral shoulder is recommended for a total slab width of 4.25 m.

As a result of extensive operating experience, the ministry has developed the following joint criteria. Joints are installed in random pattern with spacing of 3.5 m, 4.5 m, 4.0 m, and 4.3 m. Early sawcutting and adequate joint depths minimize early cracking in concrete pavement.

The design of a proper joint system in combination with good construction practices ensures the long term performance of a concrete pavement.

a) Joint Types

There are four joint types and they include the following:

a. Contraction joints
b. Construction joints
c. Isolation joints
d. Expansion joints

b) Contraction Joints

Contraction joints can be transverse or skewed. Transverse contraction joints are the dominant type in a concrete pavement and are generally designed to follow the natural cracking pattern. In a JPCP, typical transverse joint spacing should be between 24 to 30 times the pavement thicknesses. Studies in northern climates have shown that a maximum spacing of 4.5 m will significantly reduce long term transverse slab cracking. It is also important to keep slabs as square as possible with desired width to length ratios of 1 to 1.25 and a maximum of 1.5.

Contraction joint design must consider load transfer. This is the transfer of load from one side of the joint to the other. In a plain concrete slab load transfer across the joint is provided by the irregular joint face or aggregate interlock. This method of load transfer is effective until heavy truck volumes exceed 100 per day per lane. Truck volumes beyond this level cause aggregate interlock to deteriorate with time. Where traffic loadings are anticipated to exceed the limits of an aggregate interlock system, smooth epoxy coated dowel bars are added to provide mechanical load transfer. Dowels or load transfer devices also reduce slab deflections and stresses which minimize faulting and reduce corner cracking. Dowel bars are typically spaced on 300 mm centres across the pavement width.

Contraction joints are generally formed by sawcutting the slab to create a reduced cross section and initiate slab cracking at that location. The depth and timing of the sawcut is very important. Historical data has shown that a minimum depth of one-quarter of the slab thickness is necessary to ensure formation of the crack at the joint. The ministry currently specifies cutting the joint to one-third of the slab thickness. Sawcut timing is critical as late sawing will result in the development of random cracks and sawing too soon can cause spalling at the joints. Sawing should begin when raveling of the sawcut does not occur, typically between 4 and 12 hours after placement of the concrete. Transverse contraction joints should be designed to intersect catchbasins or other structure to ensure random cracks will not propagate from the structure.

c) Construction Joints

Construction joints can be transverse or longitudinal. Transverse construction joints are used at planned interruptions such as at the end of the construction day, and where unplanned interruptions suspend activities for extended times. Planned construction joints should be located at a transverse contraction joint location and should be dowelled as they are formed with a smooth vertical face.

Longitudinal joints are used to prevent random longitudinal cracking which is caused by the combined effects of load and restrained slab warping. Typically, longitudinal joint spacing varies from 3.0 to 4.0 m and is usually aligned to delineate traffic or parking lanes in urban areas.

There are two basic types of longitudinal joints; one is used when one lane is constructed at a time. With one lane at a time construction, a longitudinal joint is formed with a keyway to provide load transfer across the joint. But the ministry has eliminated the keyway joint due to construction complications. When two or more lanes are paved at a time, the longitudinal joint is generally formed by sawcutting the concrete to a depth of one-third the slab thickness.

The ministry specifies that longitudinal joints be tied together with deformed reinforcing steel bars (tiebars) to prevent lane separation. Tiebars are used in highway applications where frequent heavy loads may cause the longitudinal joints to separate. Tiebars are 15 mm in diameter, 760 mm in length and are spaced at 600 mm intervals.

d) Isolation Joints

Isolation joints allow differential horizontal or vertical movements between the concrete pavement and another structure to occur without damage to either. Isolation joints are most common in urban concrete pavements where the pavement must be isolated from manholes, or catchbasins or another pavement in a “T” intersection. Generally the joint is formed a minimum of 30 mm beyond the exposed frame in a circle or half circle to eliminate stress concentrations at corners, square or rectangular patterns.

Isolated joints are formed by placing a compressible joint filler material, 12 to 25 mm thick, for the full slab depth. The filler material should be non-absorbent and non-reactive. Isolation joints that abut an existing pavement or a “T” intersection should be constructed with thickened edges. Dowels should not be used if lateral movement must be accommodated.

e) Expansion Joints

Expansion joints are constructed in the same way as isolation joints; however, they are used to relieve compressive stresses in the pavement and are always dowelled. Historically, these joints were placed at regular intervals of 60 to 150 m to relieve compressive stress. Recent studies have shown expansion joints are detrimental to long-term pavement performance, and should not be used. Expansion joints allow adjacent contraction joints to open wider, thereby accelerating joint deterioration.

Expansion joints are now only recommended where a bridge structure is abutted.

f) Joint Sealants

Joint Sealants are used to minimize the infiltration of incompressible materials and surface water into the joint. Joints are sealed only in concrete pavements and not concrete base. Incompressible material may create point bearing pressures leading to spalling and in extreme cases "blow ups". Sealant performance is generally better in pavements with short joint spacings as this reduces stress in the sealant.

There are two types of joint sealant available, one being liquid or field molded and the other being pre-formed or a compression seal. The liquid sealant is either hot or cold poured. Pre-formed sealants are factory molded and rely on long term compression recovery for successful sealing. The ministry currently specifies the use of hot poured liquid sealant.

One of the keys to successful sealant performance is joint preparation. Joint openings should be thoroughly cleaned to assure good bonding. Most specifications now require abrasive blast cleaning of the joint surfaces when liquid sealants are used. The sealant should be recessed by 3 to 6 mm below the concrete surface to prevent damage by snowplows.

Composite Pavements

Composite pavements are those designed with a rigid concrete base and a flexible asphaltic concrete surface. Composite pavements have most of the structural characteristics of concrete pavements and the structural design procedures for composite and concrete pavement are the same. The asphaltic concrete surfacing provides improved riding qualities and noise reduction when compared with a concrete surface, and the asphalt also offers some protection to the concrete from traffic wear and weather. Typically, the concrete base is covered with a lift of high stability binder HMA and a lift of surface course HMA. The long-term performance of an experimental composite pavement is described in Reference [25].

3.3.3 FLEXIBLE PAVEMENTS DESIGN

Historically, the ministry's routine flexible pavement design methodology was experience-based and summarized as a tabular guideline. With the widespread availability of computer technology, ministry pavement design is now typically carried out using the more advanced, deflection-based procedure detailed in Ontario Pavements Analysis of Cost (OPAC 2000) [26], [27], [28] and [29], and/or the American Association of States Highways Officials (AASHTO) 1993 Guide for Design of Pavement Structures (DARWin). The AASHTO Mechanistic Empirical Pavement Design Guide (MEPDG) using AASHTOWare Pavement ME Design, is currently being calibrated for Ontario conditions with implementation expected in the near future.

Factors to be Considered

Pavement Service Life or Pavement Performance Period - The time period, in years, between new construction of a pavement and its first major rehabilitation, e.g. overlay or resurfacing, when performance has become inadequate. Table 3.3.1 lists the typical service life for new and rehabilitated pavements in Ontario.

Table 3.3.1 Service Life Experienced in Ontario

	Typical Service Lives	
	Non-Freeway	**Freeway**
New Construction:		
Asphalt Pavement	14-18	14-19
Concrete Pavement	-	25-29
Rehabilitation:		
Hot mix overlay (1 lift)	6-10	-
Mill + hot mix overlay (1 lift)	7-11	7-10
Mill + hot mix overlay (2 lifts)	10-14	10-14
Mill + hot mix overlay (3 lifts)	12-17	12-16
FDR* + hot mix overlay (1 lift)	9-12	-
FDR* + hot mix overlay (2 lifts)	12-16	-
FDR* + hot mix overlay (3 lifts)	14-17	13-17
HIR*	9	7
HIR* + overlay (1 lift)	12	10
CIR* + hot mix overlay (1 lift)	10-15	-
CIR* + hot mix overlay (2 lifts)	13-17	12-17
EAS* + hot mix overlay (1 lift)	9-13	-
EAS* + hot mix overlay (2 lifts)	11-14	7-10
Microsurfacing	8	6
Single Surface Treatment	4-8	-
Double Surface Treatment	5-9	-
Reconstruction concrete pavement	-	25-30
Rubblization + hot mix overlay (3 lifts)	-	14-17
Rubblization + hot mix overlay (4 lifts)	-	15-18

* FDR – Full Depth Reclamation; HIR – Hot In-place Recycling; CIR – Cold In-place Recycling; EAS – Expanded Asphalt Stabilization

Traffic Loading - This is given in terms of AADT and percentage of trucks. More rigorous pavement design procedures use cumulative standard 80 kN (8.2 tonnes) equivalent single axle loads (ESAL) as detailed in Section 3.2.3.

Environmental Conditions - Design must account for temperature induced stresses, moisture and frost penetration, see Sections 3.2.4 and 3.2.5.

1. ***Subgrade Soil*** - For simplicity, pavement design methods classify subgrade soils into categories based on frost susceptibility and strength, as discussed in Chapter 1.

***Drainage* -** Good granular base and surface drainage are essential in order to maintain an adequate bearing capacity and to reduce the effects of freeze-thaw cycles in the road base. Refer to Chapter 2.

***Performance of Similar Pavements* -** A history of successes and failures between similar pavements in a given area is an important guide to developing an appropriate pavement design.

***Minimum HMA Thickness* –** As per SuperPave and SMA Design Guide, Figure 3.3.4 [30].

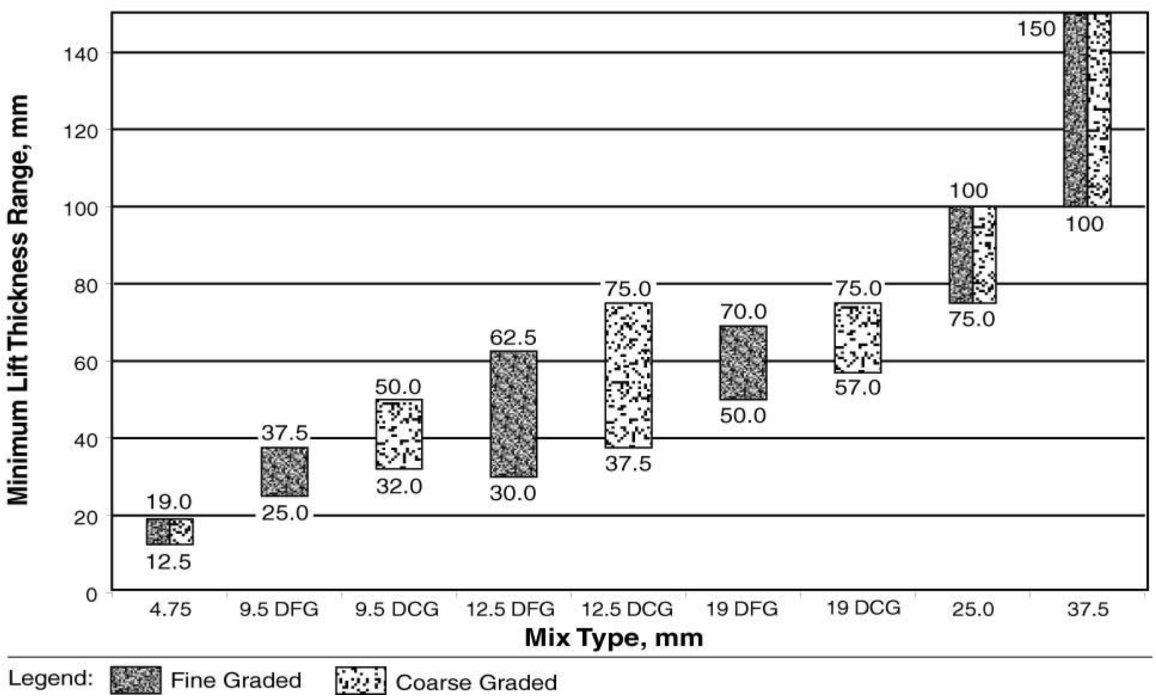


Figure 3.3.4 Recommended Minimum Lift Thickness Ranges for Dense-Graded Mixes

***Constraints and Physical Restrictions* -** Constraints include all those items listed in Section 3.2.7. Material availability is particularly critical and will vary across the province affecting the price and selection. Appropriate adjustments for local conditions are required and are generally detailed in the pavement design report. Reference shall also be made to Section 3.4 for Life Cycle Cost Analysis. Physical restrictions will have a major bearing on the feasibility and cost, see Section 3.2.7. As an example, a narrow right-of-way may require the use of urban section curb, gutter and subdrains as opposed to shoulder and ditch section.

It is important to consider constraints and physical restrictions at the pre-design or conceptual stage (ie, Scope and Cost Report [3]) of a project since they could affect the feasibility of the entire project.

Ministry Design Methods

MTO uses two different methods for flexible pavement design.

1. Routine (Empirical) Method –
    a. Experience-Based Standard Sections
    b. AASHTO 1993 – Guide for Design of Pavement Structures (DARWin)
2. Mechanistic Empirical Based Method
    a. Ontario Pavement Analysis of Cost (OPAC 2000)
    b. AASHTO - Mechanistic-Empirical Pavement Design Guide (MEPDG), AASHTOWare Pavement ME Design

<u>Routine (Empirical) Method – Experience-Based Standard Sections</u>

This method was derived from analyses of the in-service performance of historical pavement test sections and data from laboratory tests. In the mid 40’s the ministry carried out a number of plate-bearing tests and California Bearing Ratio (CBR) tests on various subgrade types. The results indicated that the actual pavement thickness was typically greater than required for structural strength. From these analyses and observations, the ministry developed the Pavement Structural Design Guidelines for flexible pavements. Over the years, these have been updated to reflect the ministry's observations and experiences [13].

The former guidelines are presented in Tables 3.3.2, 3.3.3 and 3.3.4. They indicate the surface type and thickness and the base and subbase thicknesses for various categories of facilities and for various AADT ranges and subgrade types. The tables assume 10% commercial traffic and that the foundation is a competent non-saturated subgrade. The tables also employ the concept of Granular Base Equivalency (GBE) which equates the strength of various pavement materials in terms of their thicknesses. GBE thickness is the required overall structural pavement thickness expressed in terms of an equivalent thickness of Granular A. For example, 1 mm of hot mix (HM) is equivalent to 2 mm of Granular A base which is equivalent to 3 mm of granular subbase. These and other granular base equivalencies are shown on Table 3.3.5. The thicknesses of materials shown in Tables 3.3.2 and 3.3.3 may then be varied using the equivalencies on Table 3.3.5 to derive alternative combinations of thicknesses of materials. In addition, Table 3.3.6 shows recommended asphalt surface thicknesses for various roadway facilities.

The thickness designs presented in these tables assume that granular base is constructed across the full width of the cross-section, shoulder is constructed of granular materials with or without a paved surface, and drainage of the roadbed is adequate.

Based on regional experiences in northern Ontario, it has been found that modifications must be applied to Tables 3.3.2 and 3.3.3 to account for the deep frost penetration and marginal soil conditions in these areas. Northwestern Region has modified the tables to suit its local conditions (Table 3.3.4). In Northeastern Region Table 3.3.2 is used, but with granular depths no less than those given in the table for the 2000-3000 AADT range in the southern part of the region and 3000-4000 AADT in the north of the region. The southern part of Northeastern Region is south of Highway 17 between Mattawa and Sault Ste. Marie. For higher truck percentages, designers typically move to a higher AADT range to suit the increased traffic loads.

A more comprehensive standard pavement design reference matrix was developed by ministry experts during the pavement design software development of OPAC 2000. Table 3.3.7 presents the typical pavement structure requirements in terms of GBE based on different subgrade type, traffic loadings (ESAL), and design life. This table serves as another reference for MTO experienced-based standard section design, particularly for higher traffic volume roadways.

Regardless of location, the tables provide only reference guidelines for pavement designers. Where traffic and truck volumes vary, modifications to the tables based on local experience and anticipated loadings are required. For example, if the AADT is 4,300 with 6.3% commercial traffic, it is equivalent to 2,709 AADT with 10% commercial. This is calculated by multiplying the AADT by the ratio of the % commercial: [4,300 AADT x (6.3/10)] = 2,709 AADT with 10% commercial.

Subsequent to the selection of pavement type and design thickness of layers, a detailed examination of the soils profile, construction procedures and materials availability may necessitate thickness adjustments on short local lengths of the project and possible changes in materials. When making such adjustments, consideration must be given to the design factors previously listed.

Table 3.3.2 Structural Design Guidelines for Flexible Pavements (Thickness in mm) – King's Highways and Freeways

AADT	Pavement Structure Elements	Subgrade Material: Gravels and Sands Suitable as Gran-Borrow	SANDS AND SILTS: 5-75μm <40%	5-75μm 40-55%	5-75μm >55%	Lacustrine Clays	Varved & Leda Clays
Greater than 4000 AADT	HM	130	130	130	130	130	130
	B	150-250	150	150	150	150	150
	SB	---	300-450	450-600	600-800	450	450-1100
	GBE	410-510	610-710	710-810	810-945	710	710-1145
3000-4000 AADT	HM	120-130	120-130	120-130	120-130	120-130	120-130
	B	150-250	150	150	150	150	150
	SB	---	300-450	450-600	600-800	450	450-1100
	GBE	390-510	590-710	690-810	790-945	690-710	690-1145
2000-3000 AADT	HM	90	90	90	90	90	90
	B	150	150	150	150	150	150
	SB**	---	300	450	600	450	800
	GBE	330	530	630	730	630	865
1000-2000 AADT	HM	50	50	50	50	50	50
	B	150	150	150	150	150	150
	SB**	---	250	300	450	300	450 (300-600)
	GBE	250	415	450	550	450	550 (450-650)
200-1000 AADT	HM	50	50	50	50	50	50
	B	150	150	150	150	150	150
	SB**	---	150	250	300	250	300 (250-450)
	GBE	250	350	415	450	415	450 (415-550)

Notes: All AADT Volumes refer to Present Traffic.

HM — Hot Mix Asphalt & Thickness

B — Base Thickness

SB — Subbase Thickness

GBE — Granular Base Equivalency Thickness (1 mm HM = 2 mm B = 3 mm SB)

** — Proposed subbase thicknesses may be decreased or increased respectively, for harder or softer subgrade conditions in each category, except for varved and leda clay subgrade where exceptionally large ranges are shown.

Table 3.3.3 Structural Design Guidelines for Flexible Pavements (Thickness in mm) – Secondary Highways

AADT	Pavement Structure Elements	Subgrade Material: Gravels and Sands Suitable as Gran-Borrow	SANDS AND SILTS: 5-75µm <40%	SANDS AND SILTS: 5-75µm 40-55%	SANDS AND SILTS: 5-75µm >55%	Lacustrine Clays	Varved & Leda Clays
2000-3000 AADT	HM	90	90	90	90	90	90
	B	150	150	150	150	150	150
	SB**	—	300	450	600	450	800
	GBE	330	530	630	730	630	865
1500-2000 AADT	HM	50	50	50	50	50	50
	B	150	150	150	150	150	150
	SB**	—	250	300	450	300	450 (300-600)
	GBE	250	415	450	550	450	550 (450-650)
1000-1500 AADT	CL	50	50	50	50	50	50
	B	150	150	150	150	150	150
	SB**	—	250	300	450	300	450 (300-600)
	GBE	240	405	440	540	450	540 (450-640)
500-1000 AADT	ST*	—	—	—	—	—	—
	B	150	150	150	150	150	150
	SB*	—	150	250	300	250	350 (250-450)
	GBE	150	250	315	350	315	385 (315-450)
200-500 AADT	ST*	—	—	—	—	—	—
	B	150	150	150	150	150	150
	SB**	—	150	250	300	250	300
	GBE	150	250	315	350	315	350
Less than 200 AADT	Gravel	—	—	—	—	—	—
	B	100	100	100	100	100	100
	SB**	—	150	250	300	250	300
	GBE	100	200	265	300	265	300

Notes: All AADT Volumes refer to Present Traffic.

- HM — Hot Mix Asphalt & Thickness
- B — Base Thickness
- SB — Subbase Thickness
- GBE — Granular Base Equivalency Thickness (1 mm HM = 2 mm B = 3 mm SB = 1.11)
- CL — Cold Mixed, Cold Laid or Road Mixed Mulch
- ST — Double Surface Treatment or Single Surface Treatment with Prime.
- * — Apply surface treatments 0.25 m wider than lane width.
- ** — Proposed subbase thicknesses may be decreased or increased respectively, for harder or softer subgrade conditions in each category, except for varved and leda clay subgrade where exceptionally large ranges are shown.

Table 3.3.4 Northwest Region Pavement Structural Design Guidelines for Flexible Pavement (Thickness in mm)

20 Year OPAC Loadings x 10^3	Grouping	Pavement Structure Element	Gravel	Sand			Silt		Clay	Clay & Silt	Rock & Bld(y) Gravel
	A.A.D.T.		% Passing #75 µm 0 - 10%	% Passing #75 µm 10 - 25%	% Passing #75 µm 25 - 40%	% Passing #75 µm 40 - 55%	W≤(5%> OMC)	W≤(5%> OMC)	W≥(5%> OMC)		
Urban	Multi Lane	Gran C	150	450	600	750	750	750	900	300	
		HM	130	130	130	130	130	130	130	130	
		Gran F	150	300	450	600	750	750	--	300	
4,500	5000+	Gran C	150	450	600	750	750	750	900	300	
		HM	130	130	130	130	130	130	130	130	
		Gran F	150	300	450	600	750	750	--	300	
2,000 to 4,500	3000+	Gran C	150	450	600	750	750	750	900	300	
		HM	120	120	120	120	120	120	120	120	
		Gran F	150	300	450	600	750	750	--	300	
1,000 to 2,000	2000+	Gran C	150	450	600	750	750	750	900	300	
		HM	90	90	90	90	90	90	90	90	
		Gran F	150	300	450	600	750	750	--	300	
775 to 1,000	1000+	Gran C	150	450	600	600	750	750	900	300	
		HM	50	50	50	50	50	50	50	50	
		Gran F	150	300	450	600	750	750	--	300	
500 to 775	Large % Trucks < 1000	Gran C	150	450	600	600	750	750	900	300	
		HM	50	50	50	50	50	50	50	50	
		Gran F	150	300	450	600	750	750	--	300	
300 to 500	Low % Trucks < 1000	Gran C	150	300	450	600	600	600	750	300	
		HM	50	50	50	50	50	50	50	50	
		Gran F	150	300	450	450	600	600	--	300	
< 300	Unpaved < 1000	Gran C	150	300	450	600	600	600	750	300	
		HM	--	--	--	--	--	--	--	--	
		Gran F	150	300	300	450	600	600	--	300	

Notes:

1. Pavement Design Thickness shall be determined from the 20 year OPAC loading. A.A.D.T. should only be used as a very preliminary guide. Hot Mix depths may be lowered to match "Level of Service" design.
2. W = Moisture Content; OMC = Optimum Moisture Content; HM = Hot Mix or Recycled Aspahlt on new grade; C = Cut; F = Fill
3. Granular depths include 150 mm of Granular "A".
4. Hot Mix depths may be increased by 40 mm in urban sections up to a total depth of 130 mm.
5. Granular "A" depth may be reduced by 50 mm in the unpaved grouping if future paving is anticipated.
6. Granular "A" depth may be increased where the subbase or subgrade is "clean", i.e. increase traffic maintenance tonnage per km.

Table 3.3.5 Granular Base Equivalency Factors Guideline

New Materials	
Material	**Equivalency Factor**
New (or Recycled) Hot Mix Asphalt	2.0
Granular A in Base	1.0
Granular B in Subbase	0.67
Cement Treated Material in Subbase (with Gr. A in base)	1.4
Cement Treated Material in Base (no subbase)	1.8
Bituminous Treated Material in Base (with Gr. A in subbase)	1.5
Cold Mix	1.8
OGDL	1.0

Existing or Recycled Materials	
Material	**Equivalency Factor**
Full Depth Reclamation (FDR)	1.0
Full Depth Reclamation Expanded Asphalt Stabilization (EAS)	1.6
Cold In-place Recycling (CIR)	1.8
Cold In-place Recycled with Expanded Asphalt (CIREAM)	1.8
Old HMA	1.25
Old Granular Base	0.75
Old Granular Subbase	0.5

Reconstruction Projects	
Material	**Equivalency Factor**
Old Granular Base*	0.6
Old Granular Subbase*	0.4

Notes: For design purposes Open Graded Drainage Layer (OGDL) and Surface Treatment are assumed to have no structural strength.

*Base layers in reconstruction project have lower GBE due to varieties of materials used in the past.

Table 3.3.6 Typical Thickness for Flexible Pavements

Course	Freeways and Arterials(2)	2-Lane Highways					Asphalt Courses in Composite Pavement	Bridge Decks	Asphalt Sidewalk
		Present Traffic AADT> 2000 (2)			Present Traffic AADT(2				
		>5000	2500-5000	2000-2500	1000-2000	500-1000			
(1) Surface	(6, 7, 8) 40 mm SP 12.5FC2 or SMA 12.5 or 50 mm SP 12.5FC2	(6, 7, 8) 50 mm SP12.5FC2	50 mm SP12.5FC1	40 mm SP 12.5	50 mm SP 12.5	50 mm SP 12.5	40 mm SP 12.5	40 mm SP 12.5	(4) 50 mm SP 12.5
Binder	2@50mm SP 12.5 or 1@100mm SP 25.0	1@80 mm SP 19.0 or 25.0	1@70mm SP 19.0 or 1@80 mm SP 19.0 or 25.0	1@50mm SP 12.5	Nil	Nil	40 mm SP 12.5	40 mm SP 12.5	Nil
Total Thickness	140 mm to 150 mm	130 mm	120mm or 130 mm	90 mm	50 mm	50 mm	80 mm	80 mm	50 mm

Course	Airport Runways, Taxiways and Aprons (3)	Paved Medians, Guiderail Areas, Parking Areas and Islands		Patrol Yards	Service Centers and Parking Lots	
		Built up Area & Emergency Parking (6)	No Parking		Buses and Trucks	Cars
(1) Surface	40 mm SP 12.5 FC2 or SMA 12.5	(4) 40 mm SP 12.5	(4) 50 mm SP 12.5	40 mm SP 12.5	40 mm SP 12.5	50 mm SP 12.5
Binder	50 mm SP 12.5	40 mm SP 12.5	Nil	50 mm SP 12.5	50 mm SP 12.5	Nil
Total Thickness	90 mm	80 mm	50 mm	90 mm (5)	90 mm	50 mm

Notes: SP refers to SuperPave

1 Thickness as shown with 30 mm minimum
2 Thickness selected to be determined by traffic densities, growth potential, % truck traffic, etc.
3 Design suitable for aircraft up to DC3. For airports with low traffic or mainly light aircraft use 40 mm thickness only or surface treatment. Loadings higher than DC3 require special consideration.
4 May be modified to be sandier or richer.
5 A single 50 mm course should be used for small, low traffic areas.
6 For urban freeways, 40 mm SMA 12.5 to be used.
7 For main highways carrying in excess of 5000 AADT/lane, SP12.5FC2 to be used.
8 For main highways carrying in excess of 2500 AADT/ lanes, SP12.5FC1 to be used.
A For full depth recycling use thicknesses as shown.
B Both through and approach lanes on truck inspection and freeway service stations shall have the same pavement as adjacent through traffic lanes. Minimum total thickness to be 100 mm.
C Thicknesses shown are in millimeters of compacted asphalt concrete pavement.
D Refer to MTO Surface Course Directives for surface type selections PHM-C-001.

Table 3.3.7 GBE Expert Table from OPAC 2000 for Various ESAL and Subgrade Type

Expert GBE- Weak subgrade <30MPa			
ESALs/yr	**15 years**	**18 years**	**21 years**
200000	800	850	900
500000	900	950	1000
1000000	1000	1050	1100
2000000	1100	1150	1200
3000000	1150	1200	1250
Expert GBE - Medium sub grade (30MPa-45MPa)			
ESALs/yr	**15 years**	**18 years**	**21 years**
200000	700	750	800
500000	800	850	900
1000000	900	950	1000
2000000	1000	1050	1100
3000000	1050	1100	1150
Expert GBE- Strong sub grade > 45MPa			
ESALs/yr	**15 years**	**18 years**	**21 years**
200000	600	650	700
500000	700	750	800
1000000	800	850	900
2000000	900	950	1000
3000000	950	1000	1050

Worked Example, Experience-Based Method

Design parameters: 2500 AADT average over pavement life, silt till with 40-50% silt, King's Highway.

From Table 3.3.2, the structural design consists of 90 mm hot mix, 150 mm granular base, and 450 mm granular subbase. The GBE is 630 mm.

Component	Thickness	Granular Base Equivalency (GBE)
Hot mix asphalt	90 mm	90 x 2.0 = 180
Granular base	150 mm	150 x 1.0 = 150
Granular subbase	450 mm	450 x 0.667 = 300
Total pavement thickness	690 mm	Total GBE = 630

Routine (Empirical) Method – AASHTO Pavement Design Guide 1993

The AASHTO 1993 design procedure is based on empirical performance models that were developed from the AASHO Road Test conducted in late 1950's and early 1960's [31]. The 1993 AASHTO Pavement Design Guide, also known as Design, Analysis, and Rehabilitation for Windows (DARWin) [32], procedure has been widely implemented and has been used extensively in Ontario. MTO adapted and validated the procedure for Ontario conditions, the details of which can be found in the document "Adaptation and Verification of AASHTO Pavement Design Guide for Ontario's Conditions" [10].

Major modifications to earlier pavement design methodologies include the following [1]:

- Introduction of resilient modulus to provide a rational characterization of subgrade soil and other layer material properties.
- Layer coefficients for the various materials that are related to resilient modulus, CBR and R-value.

- More objective environmental factors of moisture and temperature that replace the subjective regional factor terms previously used.
- Introduction of reliability concept to conduct risk analysis for various classes of roads.

Using AASHTO 93, the material properties of the pavement structure are characterized mechanistically using elastic theory. This involves predicting the states of stress, strain and displacement within the pavement structure when subjected to a wheel load. AASHTO 1993 recommends that the resilient modulus be established based on laboratory testing. And in most cases, soil classification is used to determine the resilient modulus. Also, seasonal moduli are identified to quantify the relative pavement damage of the seasonal effect and treating it as part of the overall design.

Pavement layer coefficients may be based on those traditionally developed and used in the original AASHTO procedure, or preferably, derived from test roads or satellite sections. Charts are available for estimating structural layer coefficients from various base strength parameters as well as resilient modulus test values.

The effectiveness of various drainage methods to direct water away from the pavement is accounted for through the use of modified layer coefficients, m, in the structural number (SN) equation along with the layer coefficient (a_i) and thickness (D_i) as shown below:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

Recommended m_i values are tabulated in the AASHTO Guide to reflect quality of drainage and percentage of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation.

Determination of the required structural numbers (SN) involves the use of a nomograph that takes into consideration:

- Estimated future traffic for the performance period
- Reliability (R) which assumes all inputs are at an average value

- Overall standard deviation (S_o) of the inputs
- Effective resilient modulus of the subgrade (M_r) and for each layer
- Design serviceability loss, $\Delta PSI = p_o - p_t$, where PSI is Present Serviceability Index (scale of 0 to 5), p_o is the initial serviceability index and p_t is the terminal serviceability index.

<u>Worked Example, AASHTO Pavement Design Guide:</u>

An example given in Appendix H of the Guide [32] is summarized to illustrate the flexible pavement design procedure.

The analysis period for the design example is 20 years, with a maximum initial service life of 15 years. Thus it will be necessary to consider planned rehabilitation alternatives over the analysis period.

Based on traffic estimates, the two-way ESAL loadings during the first year is 2.5×10^6 and the projected compounded growth rate (g) is 3% per year. The directional distribution (D_D) is assumed to be 50% and the lane distribution factor (D_L) for the three lanes in one direction is 80%. This gives the design lane one directional ESAL applications of $2.5 \times 10^6 \times 0.5 \times 0.8 = 1.0 \times 10^6$. Cumulative ESAL applications for the time periods (t) of 15 and 20 years are computed with the following equation:

$$\textit{Cumulative ESALs} = \textit{Initial year ESAL}_{\textit{(design lane)}} \times [(1+g)^t - 1] / g$$

The calculation using this equation yields 18.6×10^6 and 27×10^6 cumulative ESALs for 15 and 20 years, respectively.

Although the facility is a heavily trafficked highway, it is in a rural situation where daily traffic volumes are not expected to exceed half of its capacity. A 90% overall reliability was selected for design. This means that for a two-stage strategy (initial pavement plus one overlay), the design reliability for each stage must be 95%. Another criterion required for the consideration of reliability is the overall standard deviation (S_o); for this example an approximate value of 0.35 was used.

Environmental impacts were assessed on the basis of borehole samples and subsequent soil classifications, together with the location of the project in US Climatic Region II; i.e., wet with freeze-thaw cycling. The soil is considered to be highly active swelling clay and exposed to moisture from high levels of precipitation. As a result, a drainage system capable of removing excess moisture in less than one day is required. The duration of below-freezing temperatures is not significant enough to cause frost heaving. Detailed analysis of the soil swelling potential enabled the generation of a curve to estimate the environmental impact on pavement serviceability. The curve estimated environmental impacts associated with serviceability loss versus time. An overlay will be required before the end of the 15-year performance period due to serviceability loss associated with swelling of clay. Assuming an environmental serviceability loss ΔPSI_{ES} of 0.2, an initial service life of about 13 years is selected prior to the overlay.

Based on the traffic volume and functional classification of the facility, a terminal serviceability (p_t) of 2.5 was selected. Table 3.3.8 provides terminal and initial serviceability for different functional classifications [10]:

Table 3.3.8 Serviceability for Different Functional Classifications

Facility Types	Initial Serviceability New Construction (p_o)	Initial Serviceability Rehabilitation (p_o)	Terminal Serviceability (p_t)
Freeway	4.5	4.1 to 4.5	2.6
Arterial	4.5	4.1 to 4.5	2.5
Collectors	4.4	4.0 to 4.4	2.2
Local	4.2	3.9 to 4.3	2.0

Past experience indicates that the initial serviceability (p_o) normally achieved for flexible pavements is significantly higher than at the AASHO Road Test (4.6 compared to 4.2).

The overall design serviceability loss is:

$$\Delta PSI = p_o - p_t = 4.6 - 2.5 = 2.1$$

The effective subgrade soil resilient modulus is summarized from individual moduli determined over 24 half-month intervals to define the seasonal effects. These values also reflect the subgrade support that would be expected under the improved moisture conditions provided by the “good” drainage system. For a typical soil, the seasonal effects on M_r are as follows:

Subgrade Moisture Condition	**Subgrade Soil Resilient Modulus, M_r**
Wet	34.5 MPa(5,000 psi)
Dry	44.8 MPa (6,500 psi)
Spring-Thaw	27.6 MPa (4,000 psi)
Frozen	138 MPa (20,000 psi)

The weighted effective roadbed soil modulus, M_r for a frozen season of 1 month, a spring-thaw of 0.5 month, a wet period of 5 months and a dry period of 5.5 months, is estimated at 39.3 MPa.

Pavement layer materials characterization, based on recommended laboratory test procedures for resilient modulus (E) with corresponding structural layer coefficients (a_i values), are as follows:

Asphaltic Concrete	E_{AC} = 2758 MPa (400,000 psi)	$a_1 = 0.42$
Granular Base	E_{BS} = 207 MPa (30,000 psi)	$a_2 = 0.14$
Granular Subbase	E_{SB} = 76 MPa (11,000 psi)	$a_3 = 0.08$

These values correspond to the average year-round moisture conditions that would be expected without any type of drainage system. The effects of certain drainage methods to remove moisture from the pavement are not specified in detail; however, general definitions corresponding to different drainage levels and recommended coefficient (m_i values) for modifying structural layer coefficients are given. These values range from 1.4 for “excellent” drainage, i.e., less than 1% of the time that the pavement structure is exposed to moisture levels approaching saturation, to 0.40 for “very poor” drainage, i.e., greater than 25% of the time approaching saturation). For this example, m_i 1.0 is used.

The development of a design alternative is based on 20 year maximum initial service life; however, because of serviceability loss due to swelling, an overlay is needed to carry the traffic for the remaining 20-13=7 years of the analysis period. Using the effective subgrade soil resilient modulus of 39.3 MPa (5,700 psi), a reliability of 95%, an overall standard deviation of 0.35, a design serviceability loss of 2.1 and the cumulative traffic of 18.6 x 10^6 ESALs at 15 years, and applying Figure 3.1 from Part II of the AASHTO Guide, the maximum initial structural number (SN) is 142 mm (5.6 inch).

The thickness of each layer above the subgrade is determined by using the resilient modulus of the underlying layer. A slightly reduced serviceability loss is realized due to traffic with E_{BS} equal to 207 MPa (30,000 psi). Using an iterative procedure, SN_1 is determined to be 82 mm (3.2 inch). Thus, the asphaltic concrete surface thickness required is:

$$D_1^* = SN_1/a_1 = 82\text{ mm}/0.42 = 195\text{ mm (or 200 mm)}$$
$$SN_1^* = a_1D_1^* = 0.42 \times 200\text{ mm} = 84\text{ mm}$$

Similarly, using the subbase modulus of 76 MPa (11,000 psi), the effective resilient modulus (SN_2) is equal to 114 mm (4.5 inch) and the thickness of base material required is:

$$D_2^* = (SN_2 - SN_1^*) / a_2 m_2 = (114 – 84) / (0.14 \times 1.0) = 214\text{ mm (or 200 mm)}$$
$$SN_2^* = a_2D_2m_2 = 200 \times 0.14 \times 1.0 = 28\text{ mm}$$

Finally, the thickness of subbase required is:

$$D_3^* = (SN - (SN_2 + SN_1^*) / a_3\ m_3))$$
$$= (142 - (84+28) / (0.08 \times 1.0)) = 30 / 0.08 = 375 \text{ mm}$$

The initial structural section thickness required to carry the expected traffic for 13 years (actual service life) is:

Asphaltic concrete surface		200 mm
Granular base		200 mm
Subbase course		375 mm
TOTAL THICKNESS	=	775 mm

If it were desired to achieve a 15 year initial service life under all the foregoing condition, an increase in thickness could be used.

Example of DARWin Design

The pavement design procedures in AASHTO 1993 [32] have been computerized in the pavement design software DARWin, available from AASHTO. Below is an example of the DARWin inputs and the specified layer design output:

Flexible Design Input Parameters:

Design Lane 80-kN ESALs over Initial Performance Period	5,463,102
Initial Serviceability	4.4
Terminal Serviceability	2.2
Reliability Level	90%
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	50 MPa
Calculated Design Structural Number	**107 mm**

Specified Layer Design

Layer	Material Description	Struct. Coef. (ai)	Drain Coef. (Mi)	Thickness (Di) (mm)	Calculated SN (mm)
1	Asphalt*	0.42	1	130	55
2	Gran A	0.14	1	150	21
3	Gran B I	0.09	1	350	31
TOTAL	-	-	-	630	107

**Note: The program uses asphalt to refer to asphaltic concrete and HMA.*

Mechanistic-Empirical Based Method – OPAC 2000

Ontario Pavement Analysis of Cost (OPAC) was developed in the early 1970s as a computer-based system. It generates a range of flexible pavement design alternatives using input parameters specified by the designer. The output predicts the pavement service life and the associated life cycle cost for each design alternative. The costs include initial construction, periodic maintenance, resurfacing, salvage value and user delay costs, and vehicle emission costs. The basic design inputs are subgrade conditions, traffic projections, performance limits, available materials and their costs, range of various layer thicknesses and the maximum dollar value available for the project.

OPAC 2000 is a deflection-based method which draws heavily on the AASHTO Road Test and Brampton Road Test findings [33], [34]. OPAC 2000 incorporates an elastic layer analysis to determine the behavior and response of the pavement under load. Traffic is expressed as cumulative ESALs. Layers constructed of different materials are reduced to granular base equivalent thicknesses (GBE) by using layer equivalency factors. The steps in this design method are outlined in Figure 3.3.5.

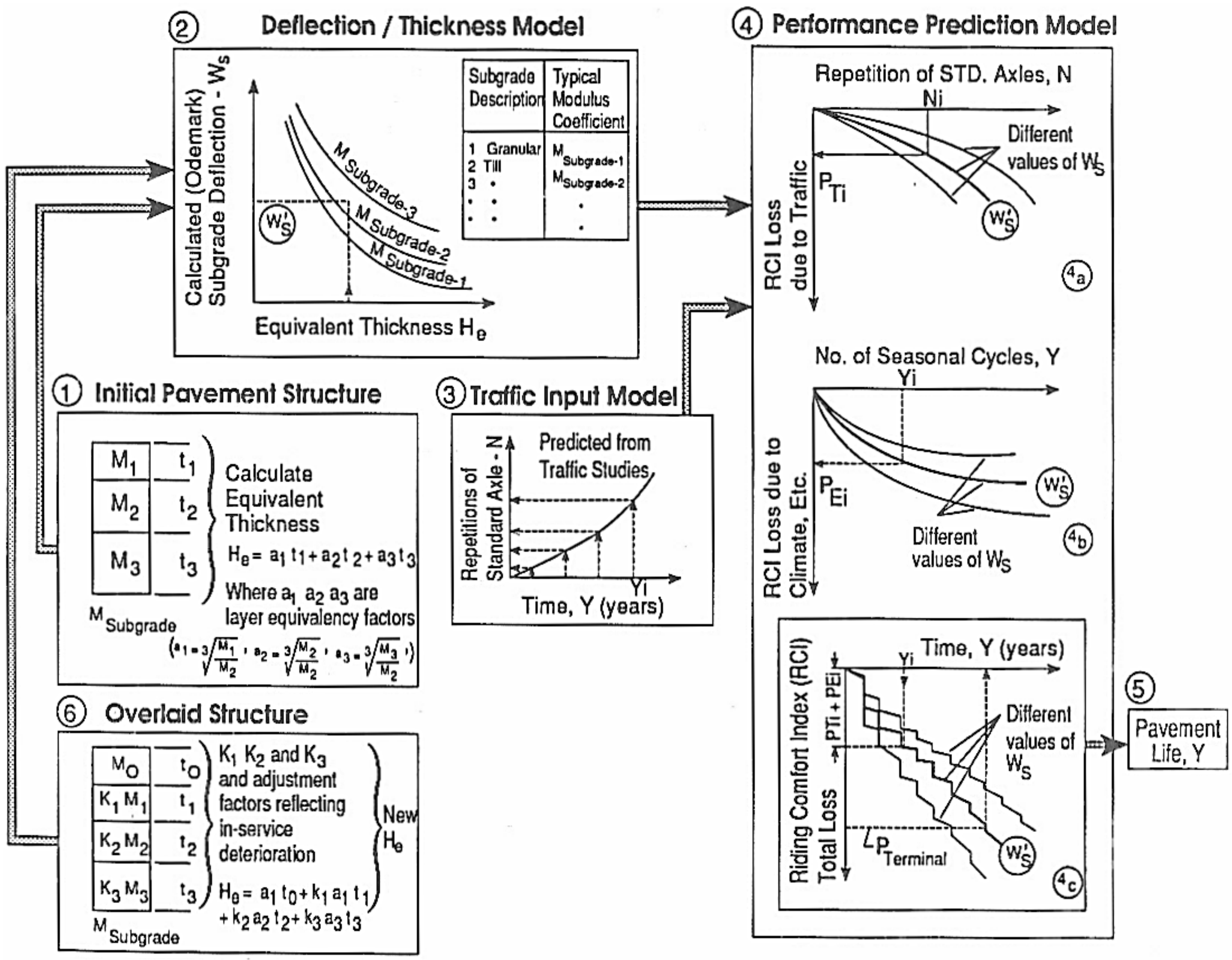


Figure 3.3.5 Flow Diagram Showing Steps in the OPAC 2000 design Subsystem

The flexible pavement design incorporates performance models, rehabilitation alternatives and enhanced economic analysis. The OPAC 2000 analysis framework is presented in Figure 3.3.6.

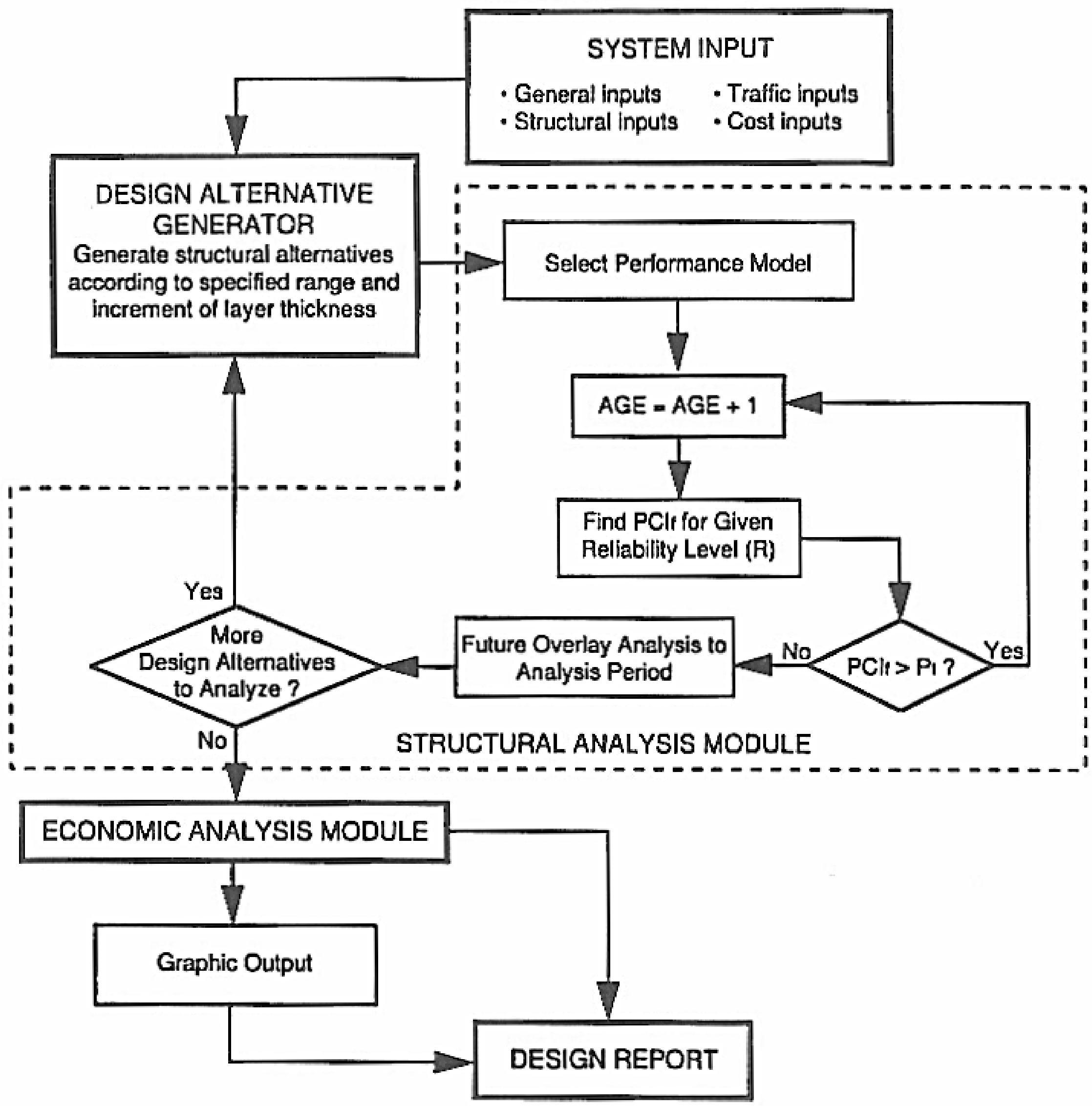


Figure 3.3.6 Framework of OPAC 2000

Structural pavement design alternatives are generated according to specified layer thickness ranges and increments. The results are organized in an n-dimensional array, where n is the number of layers. Based on pavement performance predictions and the design life span of a pavement alternative, the structural analysis is then performed. This is followed by the economic analysis which gives the life cycle cost of each design alternative. In the design report, pavement layer thicknesses, predicted pavement life and associated cost in terms of agency cost, road user cost and the total cost of each design alternative are summarized in a tabular form. The results are ranked by total cost to assist in making design decisions.

The performance prediction model for flexible pavements is expressed by the following equation:

$$P = P_o - P_T - P_E \qquad \text{(Equation 1)}$$

Where,

P = the performance index at any age, Y
P_o = the initial performance index
P_T = the performance losses due to traffic
P_E = the performance losses due to environment

The basic design factor in OPAC 2000 performance prediction model is as follows:

$$H_e = a_1h_1 + a_2h_2 + a_3h_3 + \ldots \qquad \text{(Equation 2)}$$

Where,

H_e (mm) = the equivalent granular thickness
h_1, h_2, h_3 = the actual thicknesses of the HMA, base and subbase layers
a_1, a_2, a_3 = strength coefficients of the HMA, base and subbase layer materials

H_e (mm) is also known as granular base equivalency (GBE).

This calculation of equivalent granular thickness allows the pavement to be transformed into a two layer equivalent structure, and thus the (Odemark) subgrade deflection (W_s) can be calculated as:

$$W_s = 1000 \frac{P}{2M_s Z\sqrt{1+(a/Z)^2}} \qquad \text{(Equation 3)}$$

Where

P = total load (i.e., 40kN on a dual tire)

M_2 = modulus of the equivalent granular base material (average 345 MPa)

M_s = modulus of the subgrade (MPa)

Z = $0.9He\sqrt[3]{\frac{M_2}{M_s}}$

a = radius of loaded area (ie, approximately 163 mm for an equivalent circular imprint of a dual tire)

The calculation of the Riding Comfort Index (RCI) loss due to traffic is as follow:

$$\Delta RCI_T = 2.4455\psi + 8.805\psi^3 \qquad \text{(Equation 4)}$$

Where

Ψ = $3.7239 \times 10^{-6} \times W_s^6 \times N$ (for W_s in mm)

N = number of (80kN or 18 Kip) ESAL applications

The RCI loss due to environment is expressed as:

$$\Delta RCI_E = P_o\left(1 - \frac{1}{1+\beta W_s}\right)\left(1 - e^{\alpha Y}\right) \qquad \text{(Equation 5)}$$

Where

P_o = initial RCI

W_s = as previously defined

Y = pavement age

α,β = constant

The yearly performance index of a pavement is predicted by substituting P_T and P_E in Equation 1 with ΔRCI_T and ΔRCI_E Equation 4 and 5, respectively.

OPAC 2000 uses calibrated coefficients based on a data base of observed PCI values (for "P" in Equation 1) and initial performance P_o. Three sets of coefficients are used, as shown in Table 3.3.9 below. Coefficients α and β are constants but coefficient *c* is dependent on subgrade type, ESAL and GBE. The coefficients reduce the error of pavement performance prediction in Ontario.

Table 3.3.9 Coefficients for OPAC 2000 Flexible Pavement Performance Prediction Model

Coeff.	Weak Subgrade	Medium Subgrade	Strong Subgrade
α	-0.0234596	-0.0234596	-0.0234596
β	12.7211	12.7211	12.7211
c	(0.0099*logESAL - 0.1969)*ModelGBE+(-0.000003*ESAL+78.25)	$(1*10^{-15}*ESAL^2 - 3*10^{-9}*ESAL - 0.0636)*ModelGBE +207*10^{-7}*ESAL-4.4*10^{-12}*ESAL^2+59.5$	$(-8*10^{-15}*ESAL^2+3*10^{-8}*ESAL -0.1416)*ModelGBE+1*10^{-5}*ESAL+134.02$

OPAC 2000 provides a tool for estimation based on standard engineering reliability principles. The concept of reliability in OPAC 2000 is that the pavement will provide a specified level of performance over the design period with a certain probability. The following equation is used for calculating reliability (R):

$$R = P\lfloor P_f \geq P_t \rfloor \qquad \text{(Equation 6)}$$

Where,

P_f = the serviceability index (PCI, in OPAC 2000) at a given year

P_t = the minimum acceptable serviceability level (terminal PCI)

There are two basic sources of uncertainty: (a) the idealization of design inputs, and (b) the error incorporated in the regression model. To account for uncertainty, the associated variables need to be treated as random variables instead of variables with definite values. In OPAC 2000, variables considered to contribute to the first type of error include the GBE_s (a_i), the subgrade modulus (M_s), the estimated ESAL applications (N) and the initial performance level (P_o). The model variance from regression analysis is used to account for the second type of error.

Equation 6 requires calculations of the mean variance of the dependent variable based on the distributions of independent variables. For nonlinear models, such as the one in OPAC 2000, it is often difficult to solve directly because of the integration involved in calculating probabilities. The second moment approximation method is used for calculating the variance of performance index PCI based on variances in GBE, M_s, N, and P_o:

$$\sigma_v^2 \approx \sum_X \left(\frac{\partial P}{\partial X_i} \right)^2 \sigma_{X_i}^2 \qquad \text{(Equation 7)}$$

Where

σ_v^2	=	the variance in PCI due to errors in design variables
X	=	the vector of design variables P_o, H_e, M_s and N
$\partial P/\partial X_i$	=	the partial derivative of PCI with respect to one of the design variables
$\sigma_{X_i}^2$	=	is the variance of the design variables from the inputs

The yearly PCI is:

$$PCI_f = PCI + z_R \sigma_{PCI} \qquad \text{(Equation 8)}$$

and

$$\sigma_{PCI} = \sqrt{\left(\sigma_v^2 + \sigma_M^2\right)} \qquad \text{(Equation 9)}$$

Where,

z_R = the standard normal deviation corresponding to the design reliability level.

σ_M = the standard deviation corresponding to the prediction errors due to regression, which equals 7.027432 and 4.660813 for Southern Ontario (West, Central and Eastern Regions) and Northern Ontario (Northeastern and Northwestern Regions), respectively.

The calculated PCI_f is compared with the minimum acceptable level PCI at P_t. The life of the pavement is determined as the time required for PCI_f (for given reliability) to reach P_t. The pavement life before the first overlay is the initial life. Design alternatives with an initial pavement life shorter than the specified value are discarded. For other design alternatives, the OPAC 2000 program continues to perform future overlay analyses.

When performing the life cycle cost analysis, a specified future overlay thickness is added to the pavement design at the end of each analysis cycle. The above calculations are repeated with modifications to Equation 2 as follows:

$$H_e = h_o GBE_o + \sum k_i h_i GBE_i \qquad \text{(Equation 10)}$$

Where,

h_o = overlay thickness specified in the input

GBE_o = GBE of the overlay material

h_i = layer thickness of the design alternative

GBE_i = GBE of the layer in design alternative

k_i = overlay equivalency reduction factors for HMA surfacing, base and subbase layers.

This process continues until the total life of the pavement reaches a pre-specified number of years (analysis period), normally 30 years for flexible pavements, 40 years for rigid pavements. The procedure of overlay designs on flexible pavements follows a similar process, with modifications to the generation of the design alternatives and the equation of calculating the equivalent granular thickness (H_e).

OPAC 2000 also conducts life cycle cost analysis (LCCA) of different alternatives. The user has an option to set up the funding constraints and cost components to be used in the LCCA. The purpose of LCCA is to determine the expected level of economic returns for a given size of investment. When planning investments in pavement types, it is necessary to evaluate all costs associated with the proposed project, including construction, routine maintenance and rehabilitation, and user costs.

The costs of construction, routine maintenance and rehabilitation are usually borne by the agency in charge of the road network. Road user costs are borne by the community at large in the form of vehicle operating costs (VOC), travel time costs and other indirect costs. The economic returns are mainly in the form of savings in road user costs due to the provision of a pavement with better serviceability. The user has the option to exclude user costs in the analysis or to input other values.

Example of OPAC 2000 Design

The structural analysis of a six-lane highway is provided using the OPAC 2000 flexible pavement analysis procedure. The structural inputs for the alternative design being analyzed are given in Table 3.3.10.

Table 3.3.10 Structural Design Criteria

Layers and Site Information		Other Design Criteria	
HMA concrete	50 mm	Lane width	3.75 m
HMA concrete	110 mm	Divd/Und	Undivided
Granular Base	150 mm	Initial Performance Index (P_o)	95
Granular Subbase	375 mm	Performance index after future overlays (P future)	90
Future Overlay	75 mm	Minimum acceptable Performance index (P_t)	50
Mill-off depth before future overlay	10 mm	Reliability (R)	0.9
Subgrade Strength (M_s)	34.5 MPa	COV*	0.1

*Coefficient of variance of design variables GBE, M_s and N

The traffic load anticipated is 12,500 initial AADT, increasing at a rate of 4% per year. Traffic loading consists of 17% trucks, 40% of which are two and three axle trucks, 30% four axle trucks, 20% five axle trucks and 10% six and more axle trucks. The analysis period is 30 years. The foregoing traffic inputs translate into a yearly 80 kN equivalent single axle load (ESAL) of 409,116 in the first year and 1,200,837 by year 30.

Applying the OPAC 2000 structural analysis procedure (assuming this highway is in Southern Ontario), the equivalent granular thickness (H_e) of the above pavement structure from Equation 2 is 639 mm, the Odemark subgrade deflection (W_s) from Equation 3 is 0.464 mm. The predicted yearly pavement performance is given in Table 3.3.11.

Table 3.3.11 Predicted Performance for OPAC 2000 Example

Year	PCI	Year	PCI	Year	PCI
0	93.2	11	62.5	21	72.1
1	90.2	12	59.7	22	68.8
2	87.3	13	56.8	23	65.2
3	84.4	14	53.8	24	61.4
4	81.6	15	50.7	25	57.3
5	78.8	16	88.2	26	52.8
6	76.1	17	85	27	88.2
7	73.4	18	81.8	28	85.4
8	70.7	19	78.6	29	82.5
9	68	20	75.4	30	79.8
10	65.3				

The result shows that the pavement structure will have an initial life of 15 years with 90% reliability. It requires overlays at year 15 and year 26. By the end of the 30 year analysis period, the performance index, PCI, is expected to be about 80.

In addition to the numerical solution, OPAC 2000 also provides graphical presentations to help the designer understand the process of pavement deterioration. Figure 3.3.7 is the pavement performance plot of the previous example.

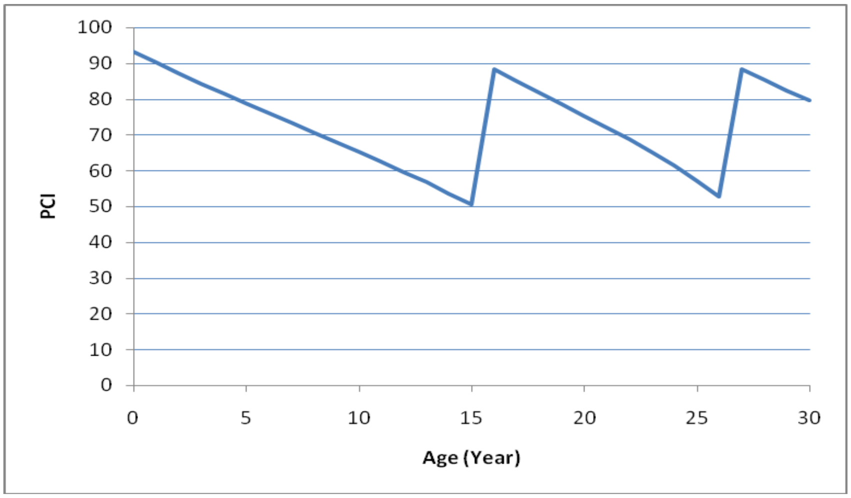


Figure 3.3.7 Predicted Pavement Performance vs. Pavement Age

<u>AASHTO 2002 - Mechanistic-Empirical Pavement Design Guide (AASHTOWare Pavement ME Design)</u>

Pavement design is moving away from empirical-based methods and moving towards a mechanistic-empirical approach. Mechanistic-empirical design is based on the principles of engineering mechanics correlated with actual field performance.

The U.S. National Cooperative Highway Research Program (NCHRP) Project 1-37A was initiated to develop the Mechanistic-Empirical Pavement Design Guide (MEPDG) based on a more rigorous approach and to develop the associated analysis software (Version 1.0) [11]. AASHTO published a “Mechanistic-Empirical Pavement Design Guide, Interim Edition: A Manual of Practice” in 2008 to assist pavement design engineers with the AASHTOWare Pavement ME Design software application.

AASHTOWare Pavement ME Design is a major enhancement to the existing American Association of State Highway and Transportation Officials (AASHTO) 1993 Guide for Design of Pavement Structures and the DARWin software tool. It includes procedures for the analysis and design of new and rehabilitated rigid and flexible pavements, procedures for evaluating existing pavements, procedures for subdrainage design, recommendations for rehabilitation treatments and foundation improvements, and procedures for life cycle cost analysis. It also provides guidance for calibrating design and rehabilitation procedures to local conditions and for developing agency specific pavement design databases.

Mechanistic-empirical design focuses on pavement performance expressed in terms of individual distresses (such as roughness, rutting, transverse cracking, faulting and punch-outs) and taking into consideration all design features and conditions that directly affect pavement performance such as materials, climate, traffic loads and construction procedures (Figure 3.3.8) [11].

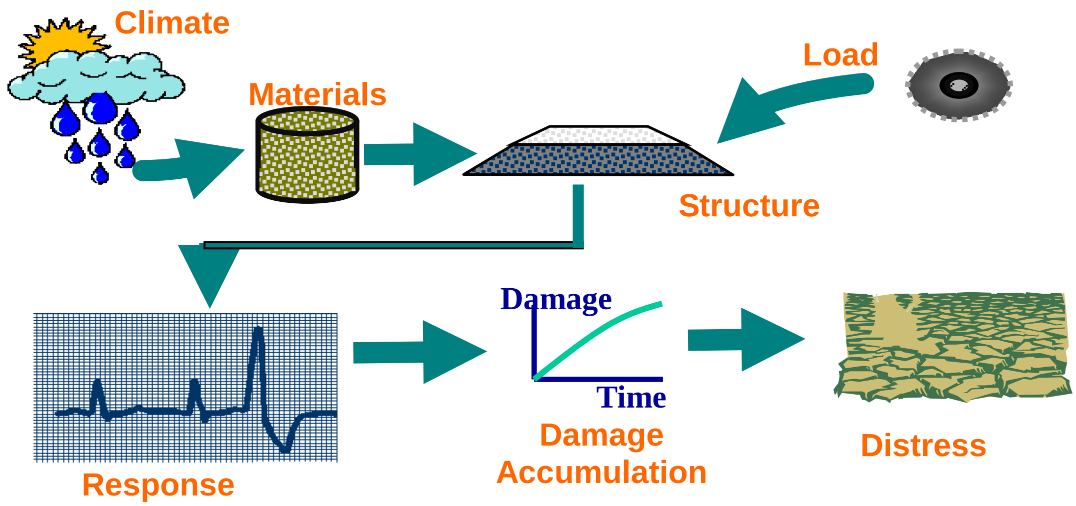


Figure 3.3.8 MEPDG Design Procedure and Analysis

The MEPDG design method is a multi-level process based on design data requirements reflecting the classification of the roadway. A significant change is the traffic input module that uses vehicle axle load spectra (axle load distribution) rather than ESALs to evaluate pavement loading during its service life. In terms of the climate module, it incorporates materials characterization and climate data through the use of an Enhanced Integrated Climate Model (EICM). By using the EICM, variations in material and subgrade properties specific to the local temperatures, humidity and precipitation may be factored into the design process.

The MEPDG design approach consists of three major stages. The three stage MEPDG design philosophy is summarized in Figure 3.3.9 [35] below:

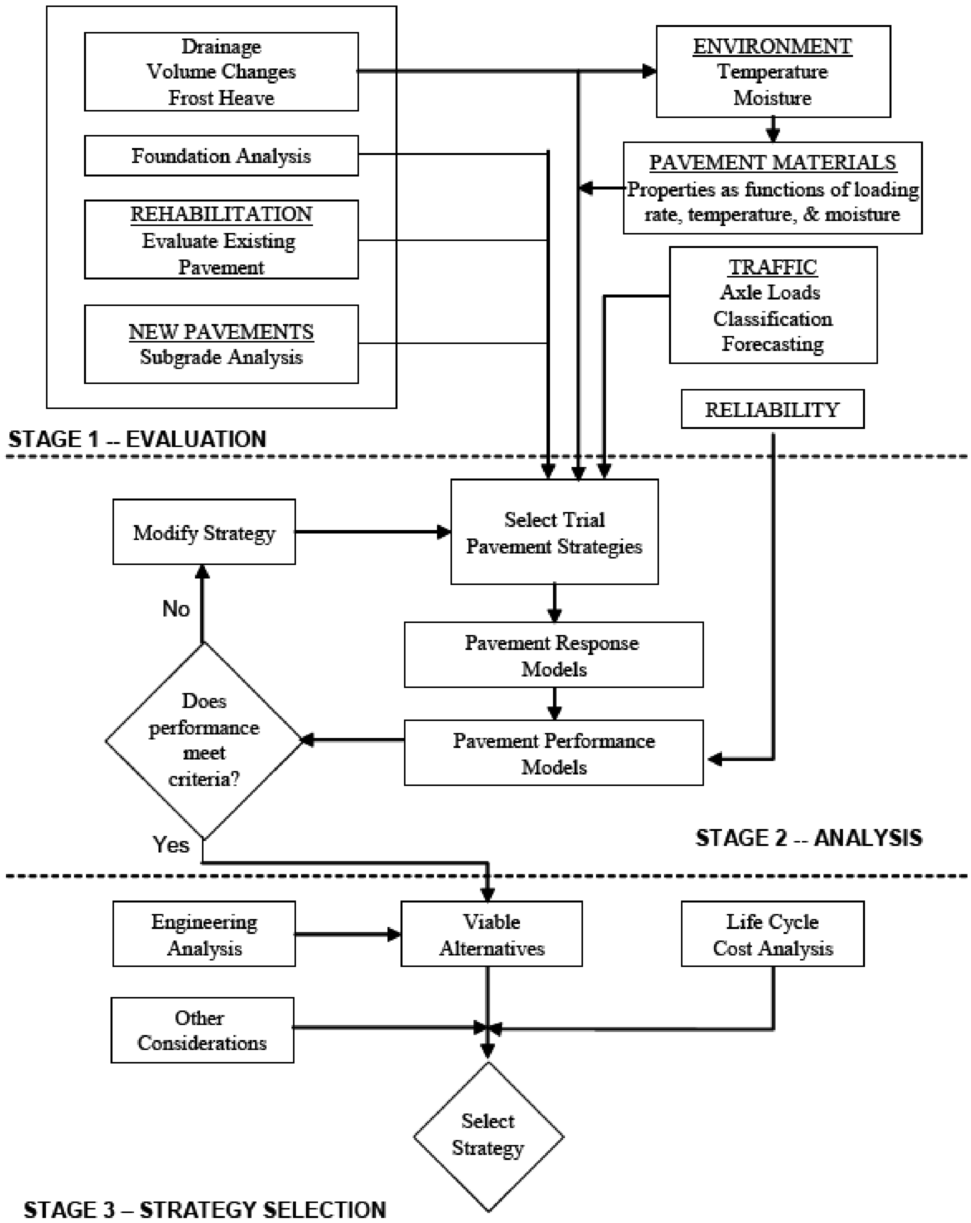


Figure 3.3.9 Conceptual MEPDG Three Stage Design Process

Stage 1 consists of the development of input values for the evaluation analysis. Stage 2 of the design process is the structural performance analysis. Stage 3 is an evaluation of structurally viable alternatives. In summary, the design process for new and rehabilitated pavement structures includes consideration of the following [35]:

- Foundation/subgrade
- Existing pavement condition
- Paving materials
- Construction factors
- Environmental factors (temperature and moisture)
- Traffic Loadings
- Subdrainage
- Shoulder design
- Rehabilitation treatments and strategies
- New pavement and rehabilitation options
- Pavement performance (key distress and smoothness)
- Design reliability
- Life cycle costs

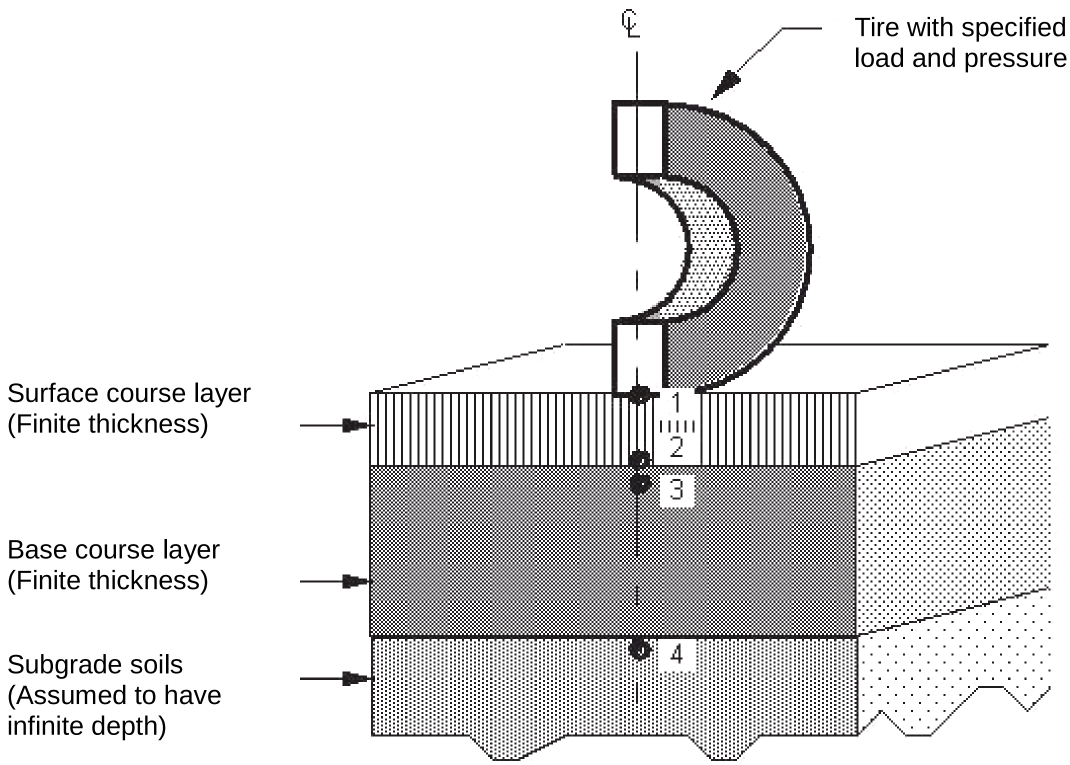
The MEPDG (AASHTOWare Pavement ME Design) requires a number of input parameters for analysis and it may not be possible to obtain all the details before designing the pavement. AASHTOWare Pavement ME Design offers a lot of flexibility and allows for different levels of design input based on the complexity of the project and the available resources [35].

There are three levels of parameter input related to traffic, materials, and environmental conditions. Level 1 input parameters provide the highest level of accuracy. They require site specific laboratory and field testing. Level 2 input parameters provide an intermediate level of accuracy and the inputs are typically user selected. The Level 2 input may be obtained from an agency database, may be derived from a limited testing program, or estimated through correlations. Level 3 input parameters provide the lowest level of accuracy. They are best estimate default values or typical averages for the region. In order to assist the pavement designers in Ontario to use this new software, a report is developed by MTO in November 2012, named " Ontario's Default Parameters for AASHTOWare Pavement ME Design – Interim Report" that provides some guidance to the designer in terms of the default input parameters. This report can be downloaded from the link below:

https://www.raqsa.mto.gov.on.ca/login/raqs.nsf/363a61d9cd2584da85256c1d0073eb7a/67c10c29044dc0a985257af400571528/$FILE/Ontario's%20Default%20Parameters%20for%20AASHTOWare%20Pavement%20ME%20Design%20-%20Interim%20Report%20(FINAL%20NOV%202012).pdf

The mechanistic aspects of pavement design are the stresses, strains and deflections within a pavement structure, while the physical aspects are the loads and material properties of the pavement structure. Empirical elements are used to determine what value of the calculated stresses, strains and deflections result in pavement failure.

The basic concept of mechanistic empirical flexible pavement design is to model the critical state or failure of the pavement structure, as illustrated in Figure 3.3.10 and Table 3.3.14 [36] identifies the typical response to loading based on different layers within a pavement structure.



1. Pavement surface deflection
2. Horizontal tensile strain at bottom of bituminous layer
3. Vertical compressive strain at top of base
4. Vertical compressive strain at top of subgrade

Figure 3.3.10 Pavement Design Critical Analysis Locations

Table 3.3.12 Critical Analysis Location in Mechanistic Pavement Design

Location	Response	Reason for Use
Pavement Surface	Deflection	Used in imposing load restrictions during spring thaw and overlay design (for example)
Bottom of HMA layer	Horizontal Tensile Strain	Used to predict fatigue failure in the HMA
Top of Intermediate Layer (Base or Subbase)	Vertical Compressive Strain	Used to predict rutting failure in the base or subbase
Top of Subgrade	Vertical Compressive Strain	Used to predict rutting failure in the subgrade

The main empirical portions of the mechanistic-empirical design process are the equations used to compute the number of loading cycles to failure. These equations are derived by observing the performance of pavements and relating the type and extent of observed failure to an initial strain under various loads.

The pavement performance in MEPDG is characterized by roughness as indicated by the International Roughness Index (IRI). Initial and terminal IRI values are specified and the roughness model is used to assess whether the pavement section can achieve the design life.

Currently, the ministry is working on calibrating the AASHTOWare Pavement ME Design to Ontario conditions to improve the accuracy of the performance prediction for local flexible pavements. More accurate distress prediction include calibration of the MEPDG models using local data and providing instructions on how to incorporate the coefficients for modifying the models to reflect local experience. Rigid pavement calibration will also be considered.

A variety of structural distresses are considered in flexible pavement design and analysis, which include [37]:

- Bottom-up fatigue (or alligator) cracking
- Surface-down fatigue (or longitudinal) cracking
- Fatigue in chemically stabilized layers (only considered in semi-rigid pavements)
- Permanent deformation (or rutting)
- Thermal cracking

For more details on the MEPDG (AASHTOWare Pavement ME Design) flexible pavement design analysis and the distress models, refer to 'Guide for Mechanistic-Empirical Design for New and Rehabilitated Pavement Structure, Part 3 – Design Analysis' reference [37].

Other Design Methods

<u>Asphalt Institute</u>

This design procedure is described in detail in Asphalt Institute (AI) Manual MS-1, 'Thickness Design - Asphalt Pavements for Highways and Streets' [38]. The method includes design charts for three flexible pavement types: full-depth asphalt, HMA surface over emulsified asphalt base, and HMA surface over granular base.

In this design, the pavement is regarded as a multi-layer elastic system. Traffic loads, expressed in terms of ESAL, are characterized as a horizontal tensile strain at the bottom of the flexible pavement layer. The result of this strain is cracking and vertical compressive strain at the top of subgrade causing permanent deformation.

The design procedure is as follows:

Step 1: Determine traffic in ESAL, subgrade modulus (M_R) which is a function of CBR, and types of surface and base material to be used. Environmental considerations are introduced by adjusting the M_R

Step 2: Enter design charts to ascertain asphalt concrete (AC) thickness

Step 3: Prepare construction staging design, if appropriate

Step 4: Prepare cost estimates for each option

Step 5: Select final design

Worked Example of AI Method

Assume the following:

- A pavement with 90 mm asphalt surface over 150 mm granular base and 450 mm granular subbase on silt till (40-50% silt) subgrade, subgrade layer coefficient M_s = 5000, 20-year analysis period.
- Cumulative ESAL, N_f, in the design lane is 1,500,000, for 12-year design service life, N = 900,000
- Initial ride comfort index (RCI), P_o of 7.5

Solution:

- 12 years of ESAL is 900,000.
- Determine M_R:
    - for a silt till, assume CBR = 7,
    - therefore, M_R = CBR x 10.3 = 72 MPa (10.3 chosen based on correlation)
- Try 150 mm A + 300 mm B = 450 mm total granular base, therefore use Design Chart VI-10 [38]. From the chart, 135 mm HMA is required.

PerRoad Software

PerRoad software is a perpetual pavement design tool developed by National Centre for Asphalt Technology (NCAT) at Auburn University, Alabama. The software is a perpetual or long-life, flexible-pavement design tool, to be used in conjunction with applicable design standards.

This design tool is a mechanistic, not empirical, procedure for design of long-life asphalt pavements. It utilizes layered elastic theory to calculate critical pavement response under applied traffic loading. The probabilistic procedure considers variability of inputs and minimizes likelihood of damage accumulation. Monte Carlo simulation is used to model uncertainty corresponding to material, loading and construction variability. This mechanistic analysis of the software determines the stresses and strains in the asphalt layers. The maximum threshold for the strain in the bottom asphalt layer must be less than 70 microstrain to control fatigue cracking and less than 200 microstrain at the top of subgrade to control structural rutting.

The design engineer defines the trial pavement structure, including the number of pavement layers, material types, material properties, variability and perpetual pavement thresholds. Climate is also part of the equation, and the engineer must specify the duration of the seasons and material properties in each season.

The PerRoad software will calculate the worse-case pavement response, as well as the probability that user-defined thresholds will be exceeded. If the performance threshold has been exceeded, the design will be judged as non-perpetual. In that case, the designer has to increase the layer thickness until the pavement responses are below the threshold. Ministry has used PerRoad to verify the perpetual pavement designs on Highway 406, 402, 401 and 7.

3.3.4 RIGID PAVEMENT DESIGN

Background

Between 1957 and 1968, concrete pavements constructed in Ontario consisted of 230 mm thick reinforced concrete slabs on 230 to 300 mm granular subbases, with transverse dowelled joints at spacings which were successively reduced from 30 m in 1958, to 21.3 m in 1960, 17 m in 1963, and 8.8 m in 1967. The major problem encountered with these designs was the increasing incidence of transverse cracks and subsequent development of spalling and faulting at joints and cracks which caused deterioration in the riding quality. No cost-effective maintenance treatment for these problems was then available so thick asphalt overlays were used to rehabilitate these pavements.

Between 1968 and 1971, concrete pavements were designed as plain 230 mm slabs with dowelled skewed joints at random spacing placed on a 125 mm cement stabilized or lean concrete base. These latter designs are still performing well under heavy traffic.

Although the performance of short-jointed plain concrete slabs on non-erodible bases in Ontario was encouraging, the costs of concrete pavements must be considered. Concrete pavement is considered as an alternative in all new and heavy traffic freeways in Ontario.

The ministry had explored several lower cost concrete pavement designs that might be, considered for two-lane highways. In 1985 the ministry published a report [39], reviewing the design, construction, and performance of four experimental concrete pavement sections and reached a number of conclusions which were being incorporated into the ministry's rigid pavement design procedure. Also, the ministry has designed a few alternative bid contracts based on the “LCC Analysis on MTO Freeway Projects” [22] and they are built in concrete pavement.

Rigid Pavement Design Considerations

Design Life - As indicated in Table 3.3.1, rigid pavements have a longer life than flexible pavements. This factor should be taken into account when assessing life cycle costs. The table indicates service life for design purposes. Actual rigid pavement life is in the 20 to 25 year range. (However, examples of concrete pavements over 40 years old are known in Ontario.) The design life of a new, reconstructed, overlaid, or restored pavement is the time from initial construction until the pavement has structurally deteriorated to the point when significant rehabilitation or reconstruction is needed. The design life is defined by the initial pavement conditions until the specified critical pavement condition has been reached at a selected level of reliability.

Traffic Loading - This is typically given in terms of AADT and percent commercial. More complex designs use equivalent single axle load (ESAL) and load spectra which is defined in Section 3.2.3.

Environmental Conditions - The design must account for temperature induced stresses as well as for moisture and frost penetration, see Sections 3.2.4 and 3.2.5.

Subgrade Soil - In rigid pavement designs, soils are divided into two major classes, erodible and non-erodible. Erosion or mud pumping beneath a rigid pavement slab is the forced displacement of a mixture of soil and water through slab cracks, joints, or edges. This leads to the formation of voids at joints and cracks and loss of subgrade support for the pavement. Erosion or pumping occurs where the soil has high fines content in the presence of free water [40].

Availability of Materials – Aggregate, hydraulic cement and steel will vary across Ontario, affecting the price and selection. Appropriate adjustments for local conditions are required, see Section 3.2.7.

Drainage - Good subsurface and surface drainage are essential in order to maintain subgrade bearing capacity and to reduce the effects of freeze-thaw cycles in the road base, refer to Chapter 2.

Performance of Similar Pavements - A history of successes and failures with other similar pavements in the area is an important guide to interpretation of the conventional design procedures.

Constraints and Physical Restrictions - Constraints include all those items listed in Section 3.2.7. Refer also to Section 3.4 for Life Cycle Cost Analysis.

Ministry Design Methods

The ministry's routine design method is described below. For more complex designs, AASHTO design method using DARWin and Portland Cement Associations (PCA) design procedure described in later sections is used as a check for structural adequacy.

Concrete pavement thickness guidelines are shown in Table 3.3.13. For each of the traffic classifications, there is a recommended minimum thickness of concrete slab. The thickness depends on the conditions of the subgrade (erodible or non-erodible subgrades) on selection of dowelled versus undowelled slabs, and on the decision to place slabs on lean concrete base (LCB), cement-treated base (CTB), open-graded drainage layer (OGDL) or directly on the subgrade. However, exceptions indicated by spaces in the table are noted in those categories where the rigid pavement design is uneconomical and/or unsuitable.

OGDL is currently used under all concrete pavements. The advantage of OGDL over CTB and LCB is that it is more flexible and is less subject to reflective cracking to the surface. Current ministry specifications dictate the use of 30 MPa hydraulic cement concrete for pavements.

Table 3.3.13 Structural Design Guidelines for Concrete Pavements

Traffic Classification		MINIMUM CONCRETE SLAB THICKNESS — mm											
		Erodable Subgrade (clays, silts and fine-grained sands)[11]						Non-Erodable Subgrade (tills and granulars)[11]					
		Undowelled[1]			Dowelled[2]			Undowelled[1]			Dowelled[2]		
	A.A.D.T.	Treat.[7] Base	OGDL[8]	Direct on Subgrade	Treat.[7] Base	OGDL[8]	Direct on Subgrade	Treat.[7] Base	OGDL[8]	Direct on Subgrade	Treat.[7] Base	OGDL[8]	Direct on Subgrade
Two-Lane	<1000		150	150					150	150			
Rural	1000 - 2000	150	175	200					150	150			
or	2000 - 3500	175	200	240	150	175	200	150	175	200			
Urban	>3500				200	225	300				175	215	250
Multi-Lane Rural					200	240	300				200	225	250
Multi-Lane Urban					225	250	300				225	250	275

NOTES:

1. UNDOWELLED means plain concrete slabs with no dowels in transverse joints (see Note 3).
2. DOWELLED means either plain concrete slabs with dowels in transverse joints (see Note 3) or reinforced slabs with dowels in transverse joints (see Note 5).
3. For plain slabs random joint spacings of 4, 5.75, 5.5, 3.6 m are recommended.
4. Skewing of transverse joints 0.6 m in 3.6 m is recommended for all transverse joints. (OPSD 509 and 510 series)
5. For reinforced slabs joint spacings of 12 m to 13.75 m are recommended.
6. All designs to have tie bars or hook bolt dowels at longitudinal joints.
7. Minimum thickness of treated base to be 125 mm of plant mix asphalt, cement-treated material, or lean concrete base.
8. Minimum thickness of OGDL to be 100 mm. A minimum of 100 – 150 mm of granular below OGDL.
9. Where paved shoulders are warranted, tied concrete paved shoulders are recommended.
   Tied concrete shoulders extend the fatigue life of the pavement equivalent to one additional inch of PCC pavement thickness.
10. Designs assume not more than 15% truck traffic.
11. For a fuller description of erodable and non-erodable subgrades refer to 'Subgrades, Subbases and Shoulders for Concrete Pavements', Portland Cement Association, Ontario Region.
12. For development of detailed structural designs for particular projects the method in 'Thickness Design for Concrete Pavements' by the Portland Cement Association, is recommended.
13. Continuously reinforced pavements are not included in these guidelines.

Other Design Considerations

The provision of a good subsurface drainage system improves overall pavement serviceability and increases pavement life and performance. The natural drainage characteristics of the granular subbase and base are augmented by the provision of an edge drainage system. These systems typically consist of a perforated plastic pipe wrapped in a knitted filter sock. The ministry specifies the use of perforated plastic pipe with geotextile wrapped trench as a drainage requirement. Refer to OPSS 405 for details.

Subsequent to the selection of pavement type and design thicknesses, a detailed examination of the soils profile, construction procedures and material availability may necessitate thickness adjustments on short local lengths of the project, and/or changes in materials. Any of these needed adjustments must be made after due consideration of the factors listed in Section 3.3.4.

Concrete pavement designs should include the following considerations:

a. Where slip-forming is used, dual grade controls should be specified to minimize surface roughness.
b. To minimize longitudinal and transverse cracking, all joints should be saw cut as soon as the concrete is in a non-plastic state.
c. Longitudinal joints and transverse joints should be cut to one-third the pavement thickness.
d. Concrete surface smoothness shall meet OPSS 350.
e. To achieve suitable surface texture, the surface shall be dragged longitudinally using burlap. This shall be followed by tining with a steel comb in the transverse direction to produce transverse grooves in the pavement for sufficient frictional resistance.

Worked Example, Routine Design Method

Design parameters: 10,000 AADT average over pavement life of 20 years on a multi-lane highway, erodible subgrade.

From Table 3.3.13 only dowelled slab joints may be used with the following thickness options:

1. 200 mm concrete pavement on 125 mm treated base (LCB)
2. 240 mm concrete pavement on 100 mm OGDL
3. 300 mm concrete pavement directly on subgrade

AASHTO Thickness Design Procedure (DARWin)

The design procedure recommended by AASHTO is based on empirical equations developed from the result of the AASHO Road Test conducted in Ottawa and Illinois, in the late 1950s and early 1960s. The computer program developed for AASHTO is called DARWin.

The basic design equation for rigid pavements is similar to the one for flexible pavements, which is based on regression analysis of the pavement performance results obtained from the AASHO Road Test. These basic equations have since been modified to include variables not considered at the Road Test. The detailed regression equation can be found in AASHTO 1993 [32]. Principal additions or modifications to the original design equations include a provision for variable load transfer at joints, and a drainage coefficient for concrete pavements. Both the flexible and rigid design procedures were modified to incorporate the concept of reliability.

The following paragraphs provide a brief overview of the design variable used in the AASHTO procedure. More detailed descriptions and design examples can be found in Reference [32].

Serviceability

The initial Present Serviceability Index, PSI, (p_o) represents the condition immediately after construction. Using current construction techniques and specifications, high quality concrete roads have initial PSI’s of about 4.7 to 4.8. In comparison, the average initial PSI of hydraulic (Portland) cement concrete pavements at the Road Test was 4.5. A higher initial PSI will reduce the required pavement thickness and will outlast a similar pavement built to a lower PSI. This rationale corresponds to the concept of serviceability to justify specifications that improve initial riding quality. Increasing the value for initial serviceability while holding constant all other input variables in the AASHTO equation demonstrates this relationship.

The terminal PSI (p_t) corresponds to the PSI at which a pavement requires some type of rehabilitation or major maintenance, where p_t varies with the functional class of road. Change in serviceability is also influenced by other parameters such as expansive or frost susceptible soils for which adjustments can be made as detailed in Appendix G of the Guide [32].

Traffic (ESALs)

Standard weights and axle configurations were used at the Road Test. In practice, there is a variety of axle weights and configurations in a mixed traffic stream. For design, these are converted to ESALs.

Design Factors

The performance of rigid pavements is influenced by many design and construction factors. In the AASHTO rigid pavement performance equation, these factors are incorporated as input variables.

Concrete Properties

There are two major properties of concrete that influence pavement performance in the Guide [32]:

S_c = concrete flexural strength determined at 28 days using third-point loading in a simple beam

E_c = concrete modulus of elasticity

Flexural strength value is based on Canadian Standards Associations, CSA A23.2-8C, Flexural Strength of Concrete. The performance equations are based on the average 28-day concrete flexural strength. Strength values measured using other test methods must be converted to the 28-day, third-point method for use in the AASHTO equation.

At MTO, the average flexural strength is used in the AASHTO procedure as reliability is now incorporated as a safety factor. Additional information may be obtained in ACPA 1993 [41].

The other concrete property used in the AASHTO design equation is the modulus of elasticity (E_c). E_c is an indication of how much the concrete will compress under load. In the rigid pavement equation, E_c has only a minor impact on thickness design or projected performance.

Load Transfer Coefficient

Each concrete pavement design will have a unique load distribution and load transfer from one side of a pavement joint or crack to the other. The use of steel dowels or continuous reinforcement in the pavement enhances load transfer. The AASHTO rigid pavement design equation includes the J-factor to account for this effect.

Edge support also influences the J-factor. Edge support is provided by widened lanes, tied or integral curb and gutter, or tied concrete shoulders. These improvements generally lead to better pavement performance.

The 1993 AASHTO procedure accounts for edge support in design. A J-factor consistent with the type of pavement and edge support is selected for design.

Subgrade Support

In rigid and flexible pavement, the subgrade eventually carries the load, and pavement performance is affected by the quality of the subgrade. In concrete pavement design, the strength of the soil is characterized by the modulus of subgrade reaction (k).

With rigid pavements, some type of subbase material is often placed on the subgrade. When this is done, the k used for design is a “composite k” (k_c) that represents the strength of the subgrade corrected for the additional support provided by the subbase.

By definition, k is determined by the plate load test. The plate load test requires placing a 760 mm diameter rigid plate on the subgrade and applying a load on the plate. The k value is found by dividing the plate pressure by displacement under a known load, expressed as MPa per metre. It can be considered in terms of pressure (MPa) on the subgrade per unit (m) of deflection of the plate.

Using the plate load test, the subgrade is modeled as a bed of springs. The value of k is analogous to the spring constant. In fact, k is sometimes referred to as the subgrade "spring constant". Once k is determined for a given subgrade soil, it is simple to calculate the deflection of the subgrade for any load.

It is important to point out that an error in the value of k has little impact on calculated pavement thickness by the AASHTO rigid pavement equation. For example, an error in the value of k of 100% only increases or decreases a typical pavement thickness by about 10 mm.

The 1986 and 1993 versions of the AASHTO Guide [31], [32] provide a loss of support factor. This factor reduces k where a loss of support is expected. A loss of support factor of 1 is equivalent to the conditions at the Road Test site.

The use of a loss of support factor is questionable, however, even though it is inherent in the AASHTO design equations. The reason is that loss of support was the primary failure mode of concrete sections at the Road Test site. It is also important to remember that a thicker pavement structural design will not, by itself, solve problems caused by water.

Coefficient of Drainage

Water is one of the primary contributors to pavement distress. Water can saturate and weaken the subgrade and subbase and pump erodible fines through pavement joints and cracks. This was the primary mode of distress and failure of rigid pavements at the

AASHO Road Test.

The 1993 Guide [32] recognized the importance of drainage by incorporating a drainage coefficient (C_d). It accounts for improved or decreased quality of drainage over those conditions at the Road Test site.

Reliability

The concept of reliability is incorporated into the 1993 AASHTO design procedure for both flexible and rigid pavements. Since it is common to both pavement types, a summary description is subsequently provided after presenting rigid pavement design, drainage design and overlay design.

Thickness (D)

At the Road Test site, slab thicknesses ranged from 62.5 to 312.5 mm. Therefore, the AASHTO rigid pavement design equation is only valid within this range. If pavement thickness calculations are unusual, it is important to check the design with another procedure such as that developed by the Portland Cement Association.

The AASHTO design procedure may generate a pavement less than 100 mm thick for light traffic. A minimum pavement thickness of 100 mm for car traffic and 150 mm for limited truck traffic is recommended. For highway applications, the ministry recommends a minimum 200 to 230 mm thickness.

After determining design pavement thickness using estimated values of the input variables or design parameters, it is advisable to check the thickness against ESAL calculations (note that the term E-18 is used in the AASHTO procedure). In practice, this recalculation will probably not significantly affect the new pavement thickness. A summary of the effects of design variables on required thickness and allowable E-18s (ESALs) is given in Table 3.3.14.

Table 3.3.14 Effect of Changes of Design Variables in AASHTO Procedure

Design Variable	Effect on	
	Required Thickness	Allowable E-18s
Increase Initial Serviceability Index, p_o	Decrease	Increase
Increase Modulus of Rupture, S_c	Decrease	Increase
Increase Modulus of Elasticity, E_c	Slight increase	Slight decrease
Increase Load Transfer Coefficient, J	Increase	Decrease
Increase Coefficient of Drainage, C_d	Decrease	Increase
Increase Modulus of Subgrade Reaction, k	Slight decrease	Slight increase
Increase Standard Deviation, S_o	Increase	Decrease
Increase Reliability, R	Increase	Decrease

PCA Thickness Design Procedure

The thickness design procedure developed by Portland Cement Association (PCA) is mechanistically based and is called PAS (Pavement Analysis Software). It was developed using theoretical studies of pavement slab behavior by Westergarrd, Pickett, and Ray. Using these theoretical studies, a comprehensive analysis of concrete stresses and deflections by a finite-element computer program was completed to develop the present procedure.

Conventional design factors of concrete properties, foundation support, and loadings are used in the procedure; as well as additional factors such as joint load transfer and concrete shoulders. The design procedure has been verified extensively by correlation with the AASHO Road Test and other road test results to study the performance of many in service pavements. More information on the development and basis of the PAC design procedure is given in Reference [42].

The procedure optimizes pavement thickness using two failure mechanisms. The first is fatigue, which requires pavement stresses due to repeated loads to be kept within safe limits to prevent fatigue cracking. The second mechanism is erosion of foundation and shoulder material, which must be limited to minimize the effects of pavement deflections at slab edges, joints, and corners. The criterion for erosion is needed to assess distress such as faulting, pumping and shoulder distress which is unrelated to fatigue. These failure criteria and the various design factors used in the thickness design analysis are discussed briefly in the following paragraphs. For full details concerning this procedure, see Reference [42]. A simplified method for concrete pavement street design is also available from the American Concrete Pavement Association [43] and in the Portland Cement Association's Information Series 184 [44].

Design Criteria

Like other construction materials, concrete is subjected to the effects of fatigue. Since the critical stresses in concrete are flexural, fatigue due to flexural stress is used for thickness design. The fatigue criterion is based on Miner's hypothesis that it is not caused by a single load, but it is damaged through repetition and cumulative loadings. The total fatigue consumed should not exceed 100%. If this level is exceeded there is a potential for fatigue cracks to develop.

The allowable number of load repetitions for a given axle load is determined based on the stress ratio. For example, a pavement subjected to flexural stress of 3.5 MPa and with a modulus of rupture of 4.8 MPa has a stress ratio f 3.5/4.8 = 0.73. Flexural fatigue research on concrete has shown that as the stress ratio decreases, the number of stress repetitions to failure increases.

Historically, mechanistic design procedures for concrete pavement were based on the principle of limiting the flexural stresses in a slab to safe values only. However, another important mode of distress is the erosion of material beneath and beside the slab. Many repetitions of heavy axle loads at slab corners and edges may cause pumping or erosion of subgrade and shoulder materials. This increase in voids under and adjacent to the slab leads to faulting (differential movement) of joints, especially in pavements with undowelled joints.

Power or rate of work associated with an axle load deflecting the slab is the parameter used for the erosion criterion. This is the product of pressure and deflection divided by the length of the deflection basin for a unit area. Development of the erosion criterion was correlated to the results of the AASHO Road Test and other studies on joint faulting.

The erosion criterion is suggested for use as a guideline. It can be modified according to local experience since climate, drainage, local factors, and new design innovations may have an influence.

In addition to fatigue and erosion criteria, studies of undowelled pavements suggest that climate and drainage are also significant factors in pavement performance. To date, these have not been incorporated into the PCA design procedure; however, investigations of the effects of climate on design and performance of concrete pavements are presented in Reference [45].

Design Factors

The PCA thickness design is based on four design factors:

a. Flexural strength of the concrete (termed modulus of rupture, MR, in the PCA procedure)
b. Strength of the subgrade, or subgrade and subbase combination (k)
c. Weights, frequencies, and types of truck axle loads
d. Design period

These design factors are briefly described as follows:

Flexural Strength of Concrete

The flexural strength of concrete is directly applicable to the fatigue criterion, which controls cracking of the pavement under repetitive truck loadings. Concrete pavements bend under axle loads, which results in both compressive and flexural stresses. Generally, compressive stresses are too small to influence design thickness, leaving flexural stress as the prime parameter.

Flexural strength determined by the third point method (CSA Standard CAN/CSA A23.2-8C) is used for design in this procedure. Strength parameters for roads and streets are generally measured at 28 days. In the design of airfield pavements, 90-day results are used because they provide a better representation of the long term strength than a 28-day tests.

If compressive strength tests are used to evaluate the quality of the concrete, the relationship between the flexural strength and the compressive strength should be determined for the mix design.

Concrete continues to gain strength with age. In this design procedure, the effects of variations in concrete strength are incorporated in the design charts and tables. The designer does not directly apply these effects but simply uses the average 28 day strength value as input.

Subgrade and Subbase Support

Subgrade and subbase support is defined in terms of the Westergarrd modulus subgrade reaction (k), which is the same parameter used in the AASHTO procedure, as described earlier. The k values are expressed as megapascals per metre (MPa/m), and are usually estimated by correlation to simpler tests such as the California Bearing Ratio (CBR). Correlation is valid as exact determination of the k value is not required. Normal variations from an estimated value will not appreciably affect pavement thickness requirements.

Generally, adjustment for seasonal variation in k is not necessary, and normal summer or fall k values are used as mean values. This was demonstrated to be a reasonable assumption at the AASHO Road Test site where reduced subgrade support during thaw periods had little or no effect on the required thickness of concrete pavements. The brief periods when k values are low during spring thaws are more than offset by the longer periods when the subgrade is frozen. Therefore, k values are much higher than assumed for design.

Where a base material is used, there will be an increase in k that should be used in design. For untreated granular material, modified k values are detailed in Table 1 of Reference [42]. Cement-treated base has been widely used in heavy-duty concrete pavements and in this case, a modified k should be used in design, as given in Table 2 of Reference [42]. Additional information on the design of concrete pavements on very stiff bases can be found in Appendix B of Reference [42].

Traffic

The PCA design procedure uses only heavy truck traffic in design calculations as the effect of lighter vehicles is negligible. Data on the axle-load distribution of the truck traffic is needed to compute the numbers of single and tandem axles of various mass expected during the design period. This data can be determined by the processes outlined in Chapter 2 of Reference [42].

Design Period

The term design period is sometimes considered to be synonymous with the term traffic analysis period. Reliability of traffic estimates for a design period greater than 20 years is low. The suggested design period for urban roads is in the order of 30 to 35 years.

The design period selected has a direct impact to thickness design. The design period dictates the number of trucks and traffic loading required for the pavement. Selection of the design period for a specific project is based on engineering judgment, economic analysis of pavement costs and service level.

Load Safety Factors

In the PCA design procedure, the axle loads are multiplied by a load safety factor (LSF). Suggested load safety factors for various facilities are:

- For multilane projects with high volumes of truck traffic, LSF = 1.2
- For highways and arterial street with moderate volumes of truck traffic, LSF = 1.1
- For residential street, and other street with low truck volumes, LSF = 1.0

Load safety factors of 1.1 or 1.2 provide a greater allowance for the possibility of unpredicted heavy truck loads and volume along with a higher level of pavement serviceability.

ACPA StreetPave Program

Recently, the American Concrete Pavement Association (ACPA) developed a software called StreetPave to replace the PAS. StreetPave Online utilizes new engineering analyses to provide recommendations for existing concrete pavements and new concrete pavement designs for city, municipal, county, and state roadways. StreetPave is specifically valuable for those involved in the design of municipal streets and roadways because the analysis tool includes updated information on current material costs and additional considerations, such as curbing. For highway design, the software tool also provides an accurate look at how a pavement will hold up under specific truck volume.

For both an existing and new pavement design, StreetPave will analyze site specific design constraints/requirements, pavement properties, and traffic characteristics. For existing concrete pavements, StreetPave will output the theoretical year in which the pavement will fail, along with the total erosion and fatigue that will occur over the user-specified design life. For a new concrete pavement analysis, StreetPave will output a design recommendation for concrete thickness, dowel bar use, and maximum transverse joint spacing. For more information and obtaining the free online version of StreetPave, visit this website: **http://www.pavement.com/StreetPave/index.asp**.

AASHTO – MEPDG (AASHTOWare Pavement ME Design)

The Mechanistic Empirical Pavement Design Guide (MEPDG), also known as AASHTOWare Pavement ME Design, is very extensive and comprehensive design tool. It includes procedures for the analysis and design of new and rehabilitated rigid and flexible pavements which are described in an earlier section. This section describes the mechanistic-empirical design procedures for new and reconstructed jointed plain concrete pavements (JPCP).

Mechanistic-empirical design focuses on pavement performance measures or criteria including joint faulting, transverse cracking, and International Roughness Index (IRI). If the trial design does not satisfy the performance criteria at a given reliability level, the design should be modified and re-analyzed until the design does satisfy the criteria. Figure 3.3.11 below illustrates the overview of the iterative design processes for JPCP.

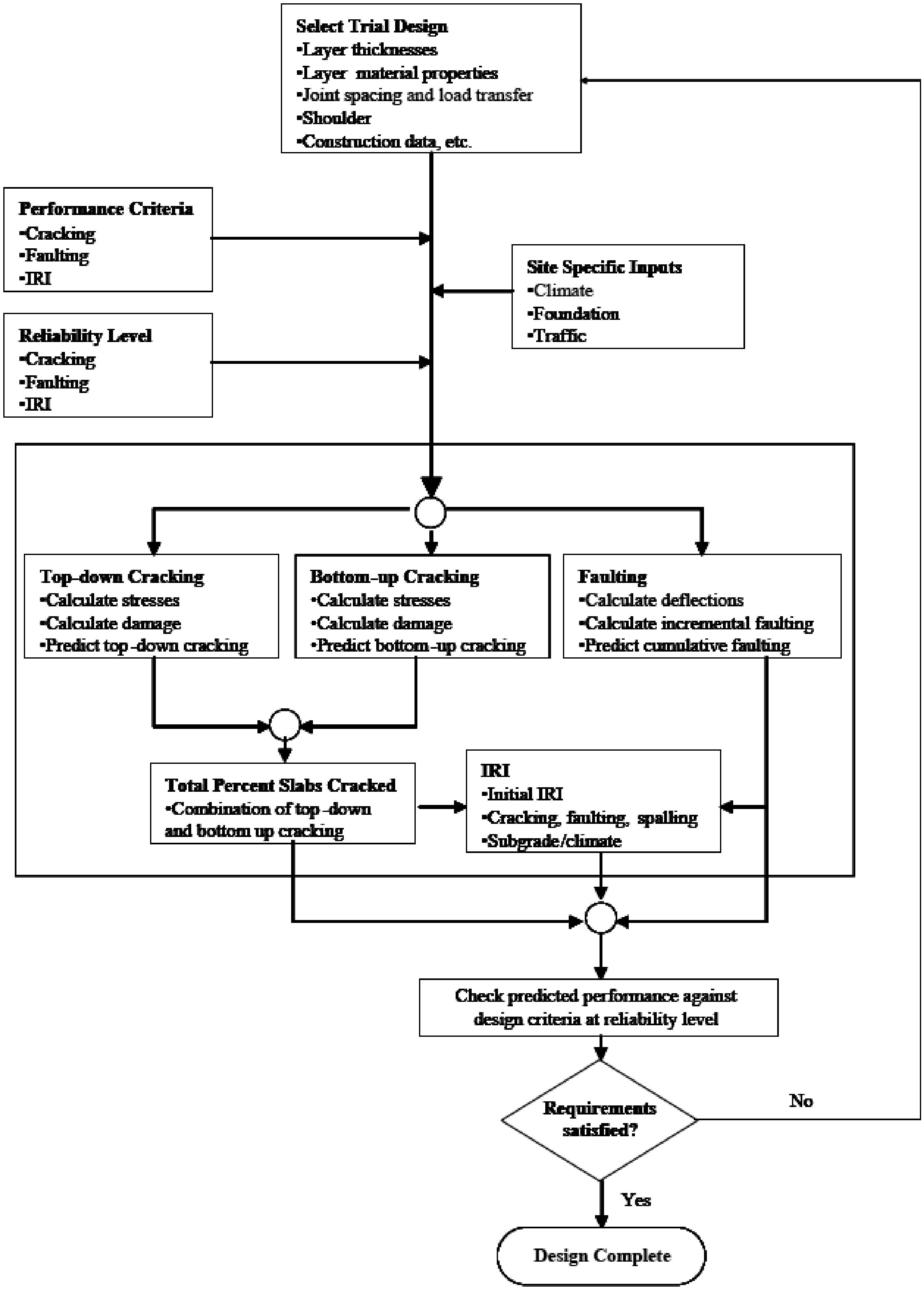


Figure 3.3.11 Overall MEPDG JPCP Design Process

The structural distresses considered for JCPC design are fatigue-related transverse cracking of concrete slabs and differential deflection related transverse joint faulting. The transverse cracking can initiate either from bottom-up cracking or top-down cracking depending on the loading and environmental condition.

Similar to flexible pavement design, the AASHTOWare Pavement ME Design for designing rigid pavement requires local condition calibration. For more information of the MEPDG (AASHTOWare Pavement ME Design) JPCP design procedure, refer to Reference [46].

Summary of Design Methods

In a detailed comparison of PCA and AASHTO Design Methods for concrete pavement, Guell [47] concludes that the PCA Method is somewhat more conservative than the AASHTO Method in the low load categories. In high load categories AASHTO is more conservative. The MEPDG (AASHTOWare Pavement ME Design) method is still under calibration for Ontario conditions.

3.3.5 OVERLAYS AND REHABILITATION DESIGN

Pavement overlays may be either HMA or concrete, although HMA overlays are more commonly used. There are four overlay scenarios:

- o Flexible over flexible
- o Flexible over rigid
- o Rigid over flexible
- o Rigid over rigid

Overlays are placed for one or more of the following reasons:

- To strengthen or rehabilitate the existing pavement and prolong its life.
- To improve safety by increasing skid resistance and/or reducing rutting and

other pavement deformation.

- To reduce tire noise.
- To improve cross-fall and surface drainage.

Ministry Design Methods

The ministry has two methods for designing and selecting materials and thickness for pavement overlays. The Simplified Method is shown on Table 3.3.15. This table relates the ministry's standard design thickness for resurfacing and in-place recycling of flexible pavements to facility type and duration of treatment. Noise reduction is achieved through the use of an open graded or dense graded mixes.

Table 3.3.15 Typical Thickness for Resurfacing and In-place Recycling of Flexible Pavements

Course	Freeways and Arterials (AADT > 5000)			2-Lane Highways					
				Present AADT > 2500			Present AADT < 2500		
	Short Term Treatment (5 years or less)	Long Term Treatment		Short Term Treatment (5 years or less)	Long Term Treatment		Short Term Treatment (5 years or less)	Long Term Treatment	
		Conventional Hot Mix Paving	In-place Recycling		Conventional Hot Mix Paving	In-place Recycling		Conventional Hot Mix Paving	In-place Recycling
New Surface	50 mm SP 12.5 FC2 (1)	40 mm SP 12.5 FC 2 or SMA 12.5 (1)	50 mm SP 12.5 FC1 (1)	40-50 mm SP 12.5 FC1 (1)	40-50 mm SP 12.5 FC1 (1)	0 or 50 mm SP 12.5 FC1 (1)	50 mm SP 12.5 (1)	50 mm SP 12.5 (1)	Nil
New Binder (5)	Nil	40-50 mm SP 12.5 or 60-80 mm SP 19.0	Nil	Nil	0 or 50 mm SP 12.5	Nil	Nil	Nil	Nil
In-place Recycled	Nil	Nil	50 to 125 mm	Nil	Nil	50 to 125 mm (2)	Nil	Nil	50 to 75 mm (2)
Milling	Depending on existing pavement thickness (3)								

NOTES: SP refers to SuperPave

1) For surface course hot mix types, refer to surface course directive PHM-C-001.
2) Only Hot in-place recycling can be used as a surface course without new hot mix overlay.
3) The need for milling will be determined at the design stage

The ministry's detailed design procedure is described below. Overlay design has been incorporated into the pavement design softwares, such as OPAC 2000, DARWin and AASHTOWare Pavement ME Design as previously discussed.

Asphalt Over Asphalt

The procedure for overlay design is described in the ministry report IR-42 [48]. It is based on the spring deflection characteristics of a typical pavement section compared with a predetermined "acceptable" deflection reading (for that combination of pavement and traffic characteristics).

Step 1: Determine AADT, convert to ESAL referred to in IR-42 [48] as Design Traffic

Number (DTN).

Step 2: From Figure 3.3.12, knowing ESAL (DTN), obtain maximum acceptable spring deflection.

Step 3: Obtain actual spring deflection from average of field reading plus twice the standard deviation.

Step 4: Enter Figure 3.3.13 or 3.3.14 with actual deflection reading on X-axis and proceed vertically to intercept allowable deflection reading, then move horizontally left to Y-axis and read overlay thickness GBE. Note: GBE of HM surface = 2.

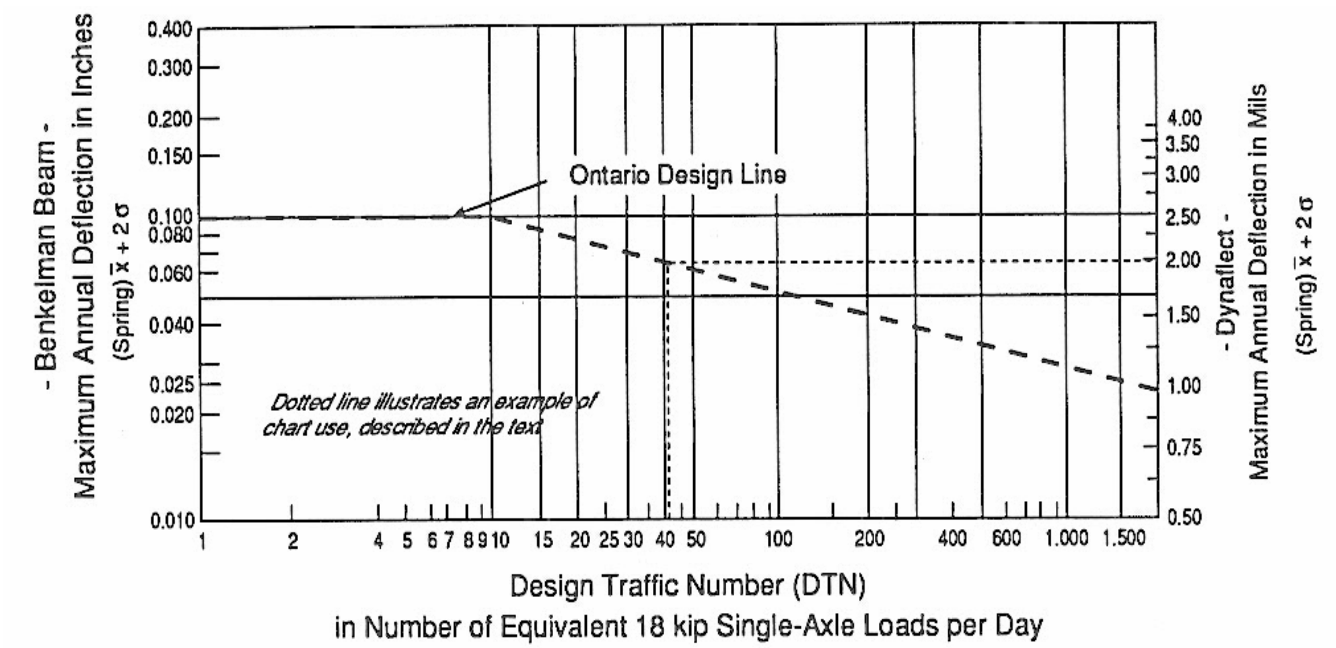


Figure 3.3.12 Ontario Pavement Overlay Design Criteria

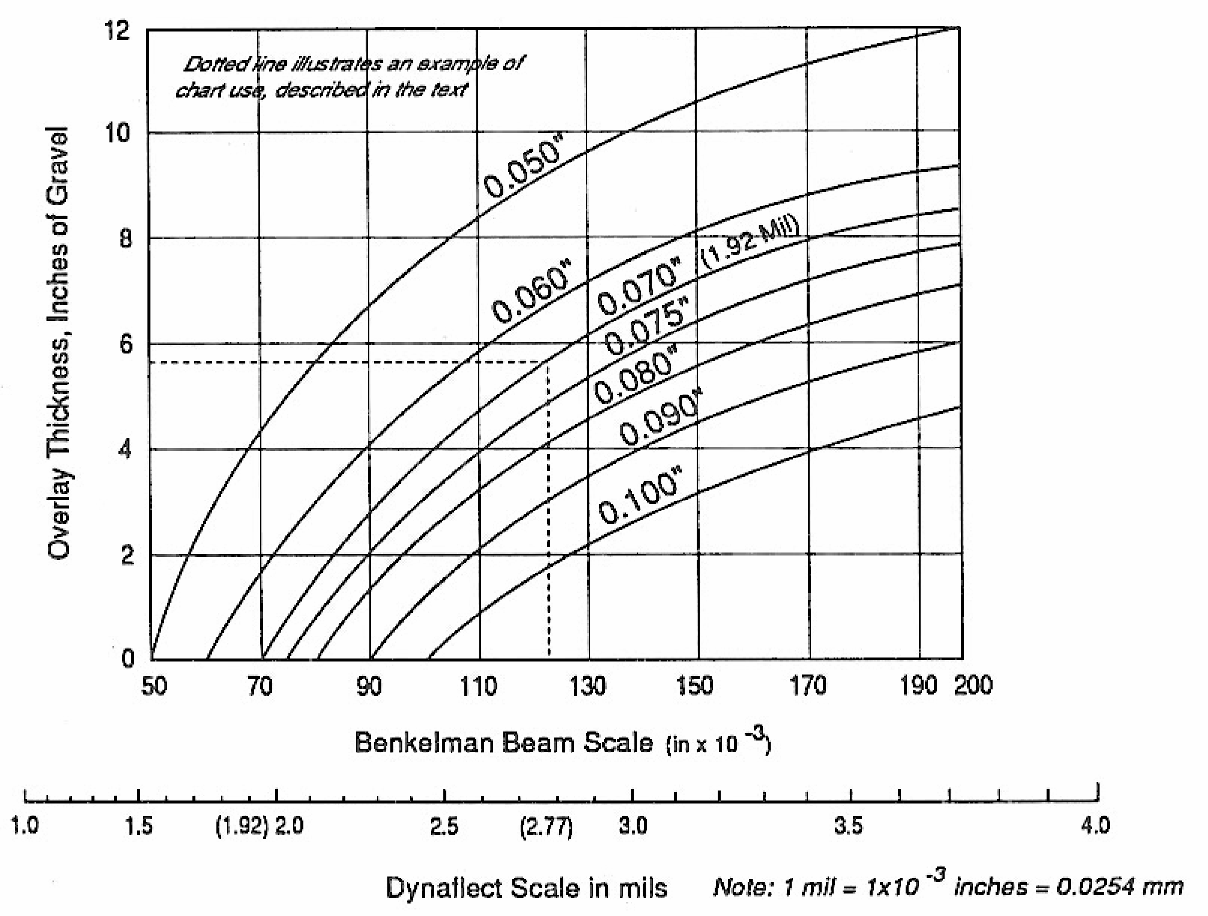


Figure 3.3.13 Pavement Thickness/Deflection Relationship (0.050 to 0.100 inches)

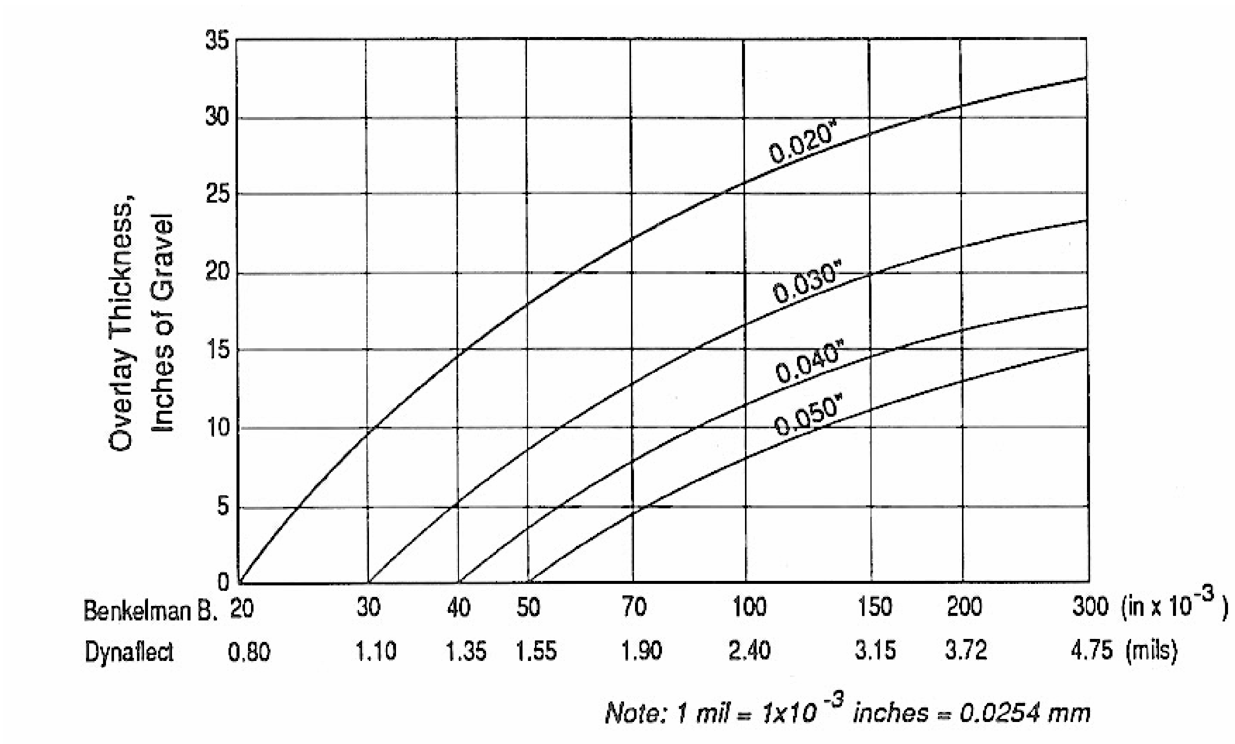


Figure 3.3.14 Pavement Thickness/Deflection Relationship (0.020 to 0.050 inches)

Worked Example **-** Design criteria: existing pavement is 150 mm HMA and 600 mm granular on silty clay subgrade of low plasticity (CL-CI), AADT 1100, 11% commercial vehicles, Dynaflect reading 1.54 mils taken in summer, with a correction factor of 1.8 corrected spring reading of 2.77 mils. From reference [48], ESAL (DTN) = 42, therefore allowable Dynaflect reading from Figure 3.3.11 = 1.90 mils. From Figure 3.3.12, using the design step procedure described above, 70 mm of HMA overlay will be required.

The Asphalt Institute Design Method from reference [49] is similar.

Asphalt Over Concrete

Flexible overlays on rigid pavement are used to improve ride and safety or to reduce noise. They may also be employed to reduce the effect of de-icing chemicals on the road structure. For this design, the thickness and quality of the overlay are determined by factors other than the structural strength. Typical hot mix overlays range from a binder and surface course for rigid pavements with minimum stepping and faulting to three or more courses consisting of SuperPave 19.0 binder, upper binder and SuperPave 12.5 FC1, SuperPave 12.5 FC2, or SMA 12.5 surface for poorly performing concrete pavements.

The AASHTO Guide [31] provides examples of other uses of overlays where the purpose is to improve the pavement strength. These examples emphasize the need to reduce the potential for reflection cracking. Methods used include using thicker hot mix overlays; breaking concrete pavement into smaller slabs; concrete rubblization; saw cutting matching joints in overlay; and absorbing crack movement in concrete by the use of a crack relief layer of granular; fiber reinforced membrane layer, or the use of SuperPave 12.5 or SuperPave 19.0. It is noted that many of these methods retard reflection cracking but do not prevent it entirely. For further details of HMA overlays using the AASHTOWare Pavement ME Design, refer to reference [50].

<u>Concrete Over Asphalt</u>

In this case the concrete overlay is designed in the conventional manner with the existing asphalt pavement being the subbase. This option is not widely used due to the significant rise in pavement surface elevation involved and its related costs.

Another rehabilitation strategy known as "whitetopping" is an overlay of concrete placed on existing asphalt pavements. Whitetopping pavements improve both the structural capacity and functional condition. Like unbonded overlays for existing concrete pavement, the major advantage of whitetopping is that a minimal amount of pre-overlay repair is required to an existing asphalt pavement. For more information on the feasibility of whitetopping asphalt pavements, see reference [32] and [51].

Concrete Over Concrete

There are three general procedures for concrete over concrete overlays. The overlay may be unbonded, partially bonded, or fully bonded. Minimum overlay thickness is in the order of 150 mm for unbonded, 125 mm for partially bonded and a maximum of 75 mm for fully bonded overlay.

Unbonded overlay design is largely independent of the existing concrete and its condition. With partially bonded and fully bonded overlays, defects in the existing concrete must be repaired since the overlay design is dependent on the characteristics and condition of the existing slab. The ministry has specified an unbonded design for Highbury Avenue (formerly Highway 126) in London, Ontario.

For further details regarding concrete on concrete overlays, refer to the AASHTO Guide [32], the Portland Cement Association Guide to Concrete Resurfacing Designs and Selection Criteria [52], and the MEPDG Guide [53].

3.4 LIFE CYCLE COSTING ANALYSIS

Highway investment, in the form of capital construction for new roads or for rehabilitation / maintenance of existing roads, represents an effort to improve safety and preserve the pavement infrastructure.

The expenditures for a highway infrastructure are spread over the life of the road and are referred to as the Life Cycle Costs (LCC). When comparisons are made between options, whether for capital construction or routine maintenance, it is necessary to relate all costs to a common benchmark. To facilitate this, the influence of interest or discount rate as well as the inflation must be considered. Procedures for doing this are given in section 3.4.2. It is important to select an analysis period that encompasses at least one rehabilitation period. A minimum 30-year period is recommended for rigid pavements and deep-strength flexible pavements. A period of 20-25 years is recommended for conventional flexible pavements.

The procedure for analyzing pavement costs is given in Section 3.4.2 and in Economic Analysis Elements, RR-20 1 [54]. Also, MTO report MERO-018 illustrated the life cycle costing analysis (LCCA) for MTO freeway project [22].

3.4.1 COST/BENEFITS FACTORS

Costs

The major cost components evaluated for a pavement design strategy are:

- ***Initial Construction Costs*** – developed for the selection initial designs. Costing for each design is based on local unit prices extrapolated to the anticipated construction date.
- ***Rehabilitation Costs*** – associated with future rehabilitation or other upgrading when the Pavement Condition Index (PCI) reaches a specified minimum level of

acceptability. This PCI minimum level is dependent on road function and classification.

- ***Maintenance Costs*** **–** associated with items directly affecting pavement performance. Annual maintenance costs increase with pavement age.
- ***Salvage Value*** - for materials that can be salvaged and reused, taking into account deterioration in material quality over time. It should include costs associated with removal of material for reuse. The total salvage value may be negative if material has to be hauled away to make room for new road construction.
- ***User Costs*** **–** related to vehicle operating costs, user travel time costs, traffic delay costs due to the construction, accident costs and discomfort costs. User costs are affected by riding condition, vehicle speed as well as safety features such as site distance. These costs are difficult to quantify.

Benefits

The benefits of a road network improvement program are either direct benefits or indirect.

- ***Direct Benefits*** - result from any improvement to the road network such as reduced travel times and improved accessibility to property and building development adjacent to the improvement works.
- ***Indirect Benefits*** **–** are those associated with salvage of materials, greater than anticipated life expectancy, and lower construction bids. It is important not to double-count these items.

Analysis Period

A general guideline for selecting the length of analysis period is that it should not extend beyond the period of reliable forecasts. The analysis period is a specific time period over which all costs are included to compute the Present Worth (PW) and allow comparison of pavement design alternatives. The ministry recommends that a 50-year analysis period be used for high-volume roadways with greater than 1 million Equivalent Single Axel Loads (ESALs) per year (current or projected within 5 years), for all freeways and 400 series highways, and for all concrete pavements (any facility type).

Discount Rate – Inflation and Interest Rate

A discount rate is used to reduce future expected expenditures (or benefits) to present day terms. It should not be confused with an interest rate, which is associated with borrowing money. A discount rate brings all future expenditures to the same time reference point, and it takes into account inflation and interest rate.

The discount rate reduces the impact of future costs, especially over long time periods. Low discount rates favor those alternatives that combine large capital investments with low preventive strategy costs, whereas high discount rates favor the reverse.

In 1998, the Ministry of Finance set the discount rate used by MTO LCCA at 7% to convert future costs to present-day costs. The Ministry of Finance revised discount rate in May 2003 to 5.3 percent, and once again revised it to 5.0 percent. It is expected that the Ministry of Finance will revise the discount rate periodically. A standard deviation of 0.52 percent can be used for probabilistic analysis. [22]

3.4.2 METHODS OF ECONOMIC EVALUATION

There are a number of economic models that may be used to incorporate costs and benefits associated with different pavement designs. In order to make realistic comparisons, it is necessary to identify the differences in the worth of money over time. It is reflected in the compound interest rate equations that incorporate discount rate and inflation rate. Economic Analysis for Highways by Winfrey [55] provides details of the methods referred to below.

Present Worth Method

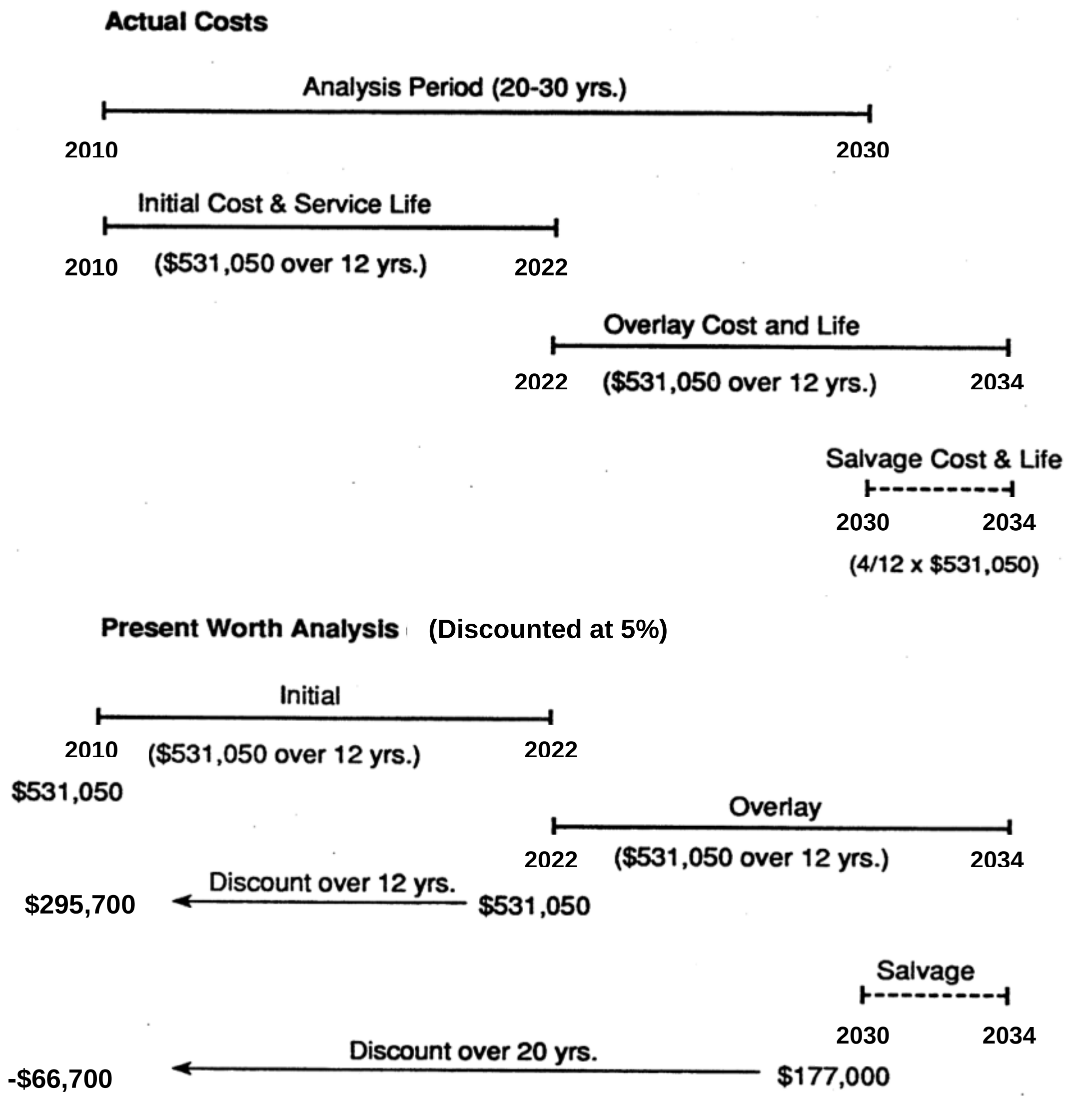
Present Worth method attempts to forecast tomorrow's purchasing value of today's funds. It is widely used in the transportation field and is quite applicable to the pavement sector. This method is preferred by the ministry. The Present Worth Method considers costs, benefits, or cost and benefits together. It discounts of all future sums to the present, using an appropriate discount rate. Detailed procedures and equations are given in Reference [22], [23] and [55].

Simplified Procedure

The ministry uses the simplified (deterministic approach) procedure for routine life cycle cost analysis. The input values such as initial construction costs are single values representing the mean value and do not account for variability or uncertainly of the inputs. A worked example is shown in Figure 3.4.1. This analysis includes only initial rehabilitation, first overlay costs, and salvage return costs.

a. Obtain costs for various rehabilitation schemes from the Estimating Office.
b. Based on the anticipated life expectancy (service life) of each scheme, assign an initial life cycle (time to first overlay) in years to each scheme.
c. Design and cost a first overlay treatment for each scheme which would be applied at the end of the life cycle.
d. Using the discount rate prescribed by the Ministry of Finance (5%) to calculate present worth of the overlay treatment. The discount rate accounts for the nominal rate of inflation and the cost of borrowing money.
e. Calculate the salvage return years and factored salvage costs based on the excess life of the schemes beyond the 20-year analysis period. Discount the salvage return costs to their present value from the end of the analysis period.
f. The present worth cost of an overlay plus the initial rehabilitation costs less the discounted salvage costs are calculated to give the life cycle cost. An illustration of present worth life cycle costs over 30 years is given in Figure 3.4.2 in which

initial, maintenance, resurface and salvage costs are shown for five alternative freeway pavement designs.



Total Life Cycle Cost

Initial Rehabilitation Cost	+	Present Worth Overlay Cost	–	Present Worth Salvage Cost
$ 531,050	+	$295,700	–	$66,700
	=	$760,050		

Figure 3.4.1 Present Worth Method, Work Example

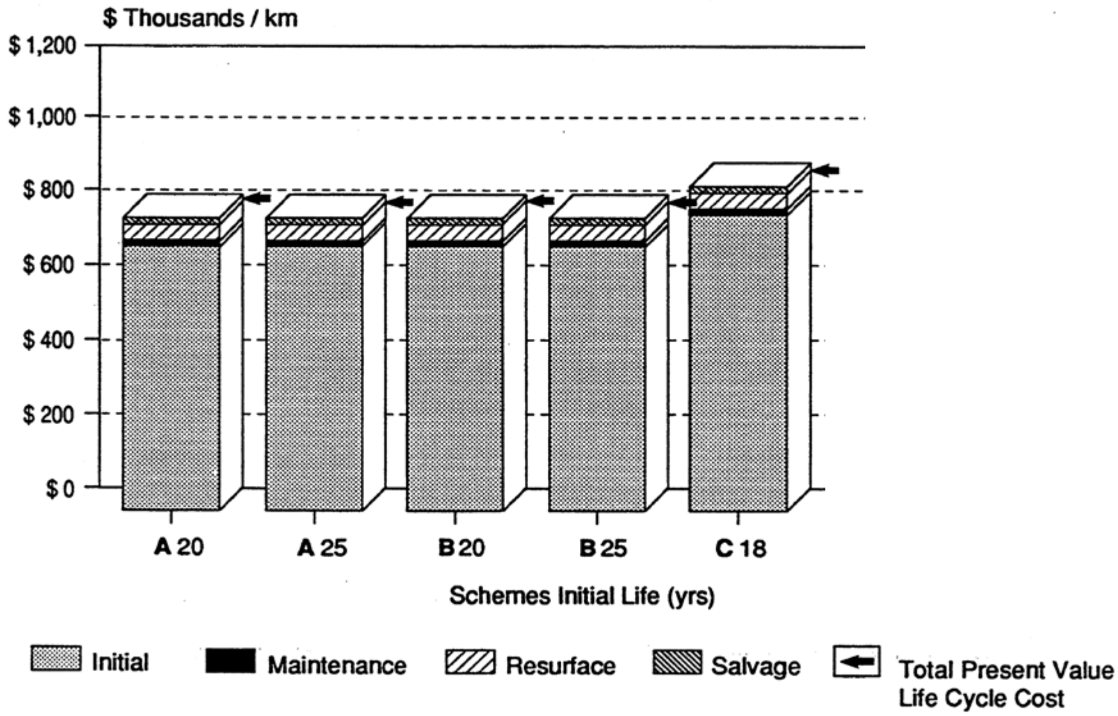


Figure 3.4.2 Present Worth Life Cycle Costs (30 years)

<u>Probabilistic Procedure</u>

Probabilistic life cycle cost analysis is carried out on more complex projects particularly when alternative bids maybe considered between rigid and flexible pavements. A probabilistic-based LCCA uses mean and standard deviation values to define a probability distribution for a given input parameter. One input is allowed to vary according to a defined distribution (e.g., normal, uniform, exponential) while holding all other inputs at their mean value and computing the present worth for each input. This is repeated many times and the present worth computed each time until sufficient runs have been made to establish the expected distribution of present worth. The range and shape of this distribution represents the impact or sensitivity of one input on the present worth. [22]

A probabilistic LCCA approach should be used for high-volume roadways with greater than 1 million ESALs per year, for all freeways and 400 series highways, and for all concrete pavements, since it more realistically represents the uncertainty and variability of highway construction and pavement performance. [22]

The probabilistic approach can be carried out using a statistical software package such as Crystal Ball® formally developed by Decisioneering Inc, and now it is part of Oracle product solution. Normal probability distributions are assigned to the following inputs, using the recommended mean and standard deviation values for:

- Discount rate
- Unit costs of individual pay items
- Service lives of each initial pavement type
- Service lives of each rehabilitation type.

As Crystal Ball® performs up to thousands of iterations, a record is kept of how many times each pavement alternative has the lowest total LCC. This allows selection of the alternative with the lowest LCC and the least risk [22]. Refer to References [22] and [11] for more details on the recommended input values (cost and standard deviation) for probabilistic LCCA. An example of probabilistic analysis is shown in Figure 3.4.3 to Figure 3.4.5.

Existing Pavement
125 mm of old HMA
150 mm of Granular Base
300 mm of Granular Subbase
Structural Number = 68 mm

Alternative A - Mill and Overlay
Mill 80 mm and replace with 2 lifts of HL 4
Existing 45 mm of old HMA
150 mm of Granular Base
300 mm of Granular Subbase
Structural Number = 81 mm
Expected Life = 8 years
Grade Raise = 0 mm

Alternative B - Pulverize and Pave
Place 100 mm of HL 4 - Two Lifts
250 mm of pulverized old HMA and Base
100 mm of new Granular Base
50 m of old Granular Base
300 mm of Granular Subbase
Structural Number = 81 mm
Expected Life = 12 years
Grade Raise = 100 mm

Geometric Details		
Length =	1000	m
Width =	3.75	m
Shoulder Width =	3	m
HMA Unit Weight	2.4	t/m3
Granular Unit Weight	2	t/m3
Milling Depth	80	mm
Alternative A Overlay	80	mm
Alternative B Overlay	100	mm
Granular Base Depth	100	mm
Shoulder Grade Raise	200	mm

Service Life and Discount Rate		
Service Life - Alternative A	8	years
Service Life - Alternative B	12	years
Service Life - Rehabilitation	8	years
Analysis Period	20	years
Discount Rate Trial	6	%
Discount Rate Mean	6	%
Discount Rate Std. Dev.	1.5	%

Initial Cost - Alternative A	$64,987.50
Initial Cost - Alternative B	$63,487.50
Rehabilitation = Mill 80 mm and replace 80 mm	$64,987.50

Alternative A - Life Cycle Costs			
	Life	Initial Cost	Present Worth
Construction	8	$64,987.50	$64,987.50
Rehabiltation 1	8	$64,987.50	$40,773.96
Rehabiliation 2	8	$64,987.50	$25,582.09
Salvage	-4	($32,493.75)	($8,025.26)
Total Present Worth		**$162,468.75**	**$123,318.29**

Alternative B - Life-Cycle Costs			
	Life	Initial Cost	Present Worth
Construction	12	$63,487.50	$63,487.50
Rehabiltation 1	8	$64,987.50	$32,296.80
Salvage Value	0	$0.00	$0.00
Total Present Worth		**$128,475.00**	**$95,784.30**

Figure 3.4.3 Probabilistic LCCA Example Input Parameters

Summary:

Display Range is from $16,970.96 to $106,552.80 $
Entire Range is from $16,563.94 to $112,285.95 $
After 1,000 Trials, the Std. Error of the Mean is $544.21

Statistics:	Value
Trials	1000
Mean	$64,673.69
Median	$64,293.79
Mode	---
Standard Deviation	$17,209.59
Variance	$296,169,956.55
Skewness	-0.04
Kurtosis	2.61
Coeff. of Variability	0.27
Range Minimum	$16,563.94
Range Maximum	$112,285.95
Range Width	$95,722.01
Mean Std. Error	$544.21

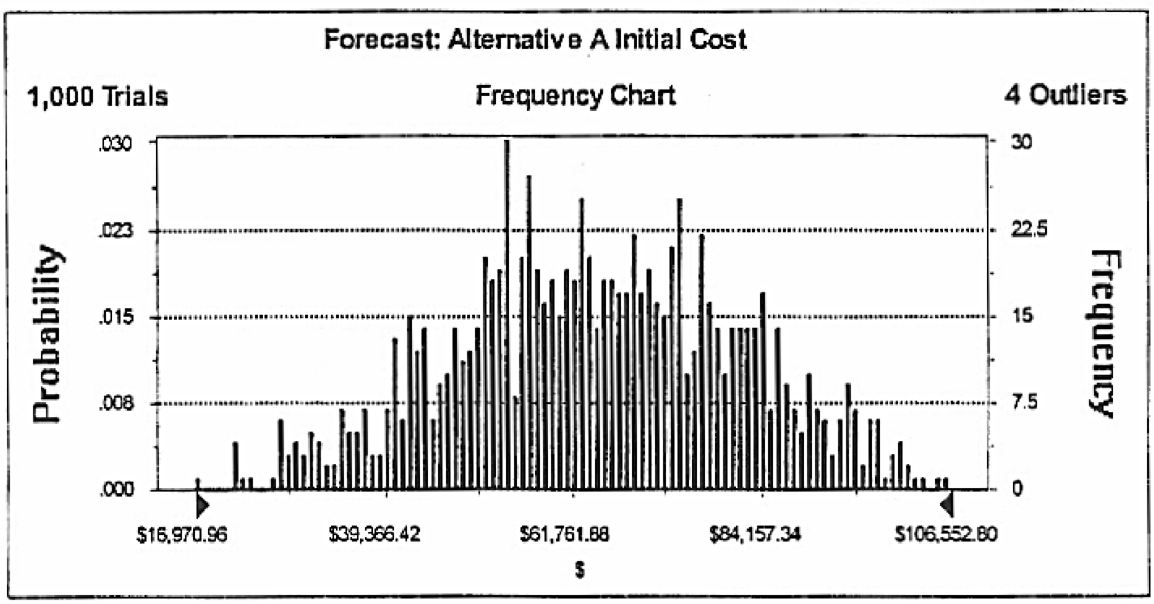


Figure 3.4.4 Probabilistic LCCA Example Alternative A – Initial Cost

Summary:

Display Range is from $50,065.35 to $77,034.42 $
Entire Range is from $48,659.74 to $78,498.19 $
After 1,000 Trials, the Std. Error of the Mean is $165.41

Statistics:	Value
Trials	1000
Mean	$63,526.33
Median	$63,530.29
Mode	---
Standard Deviation	$5,230.60
Variance	$27,359,148.62
Skewness	-0.02
Kurtosis	2.89
Coeff. of Variability	0.08
Range Minimum	$48,659.74
Range Maximum	$78,498.19
Range Width	$29,838.46
Mean Std. Error	$165.41

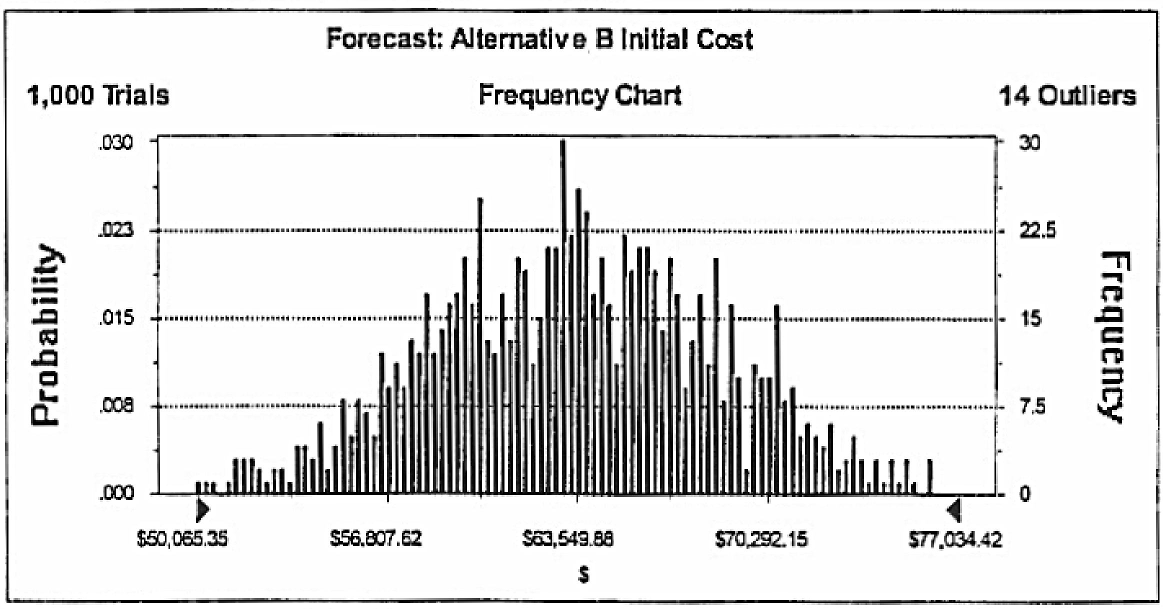


Figure 3.4.5 Probabilistic LCCA Example Alternative B – Initial Cost

The probabilistic LCCA example above indicated the initial costs calculated using deterministic (Figure 3.4.3) and probabilistic (Figure 3.4.4 and Figure 3.4.5) methods are not the same due to the variability of input parameters. Therefore it is important to utilize the probabilistic LCCA for any high risk projects to realistically represent the uncertainty and variability of highway construction and pavement performance. However, since the ministry adopted the use of alternative bid contract where the lowest bid will dictates the pavement design, the needs for calculating LCCA using probabilistic is diminished.

Equivalent Uniform Annual Cost Method

The equivalent uniform annual cost method combines initial capital costs and recurring future expenses into equal annual payments over the analysis period.

The disadvantage of this method is that it does not include benefits in the evaluation. Consequently, comparisons between alternatives must be made on the basis of costs alone, with the inherent assumption that they have equal benefits. For details see Reference [56].

Benefit-Cost Ratio Method

Using this method, the benefits associated with a pavement design alternative are expressed in present worth dollars and compared against the present worth of costs. Benefit-cost ratio is also expressed as the ratio of the equivalent uniform annual benefits to the equivalent uniform annual costs. For details see Reference [56].

MTO's Highway Element Investment Review (HEIR) guidelines are available to assist highway engineers and designers determine the project elements to include in a given construction contract on the basis of anticipated benefits and costs. A systematic methodology is provided for selecting and prioritizing improvements using principals of economic analysis.

3.4.3 THE ONTARIO PAVEMENT ANALYSIS OF COSTS

The OPAC 2000 system is described fully in RR-201, Economic Analysis Elements [54], the fundamentals of which are described in Section 3.3.3. The ministry has developed a series of algorithms to cover the various cost inputs scenarios. Cost information for various pavement design strategies are presented in the format shown in Table 3.4.1 which is an example of the OPAC 2000 computer output.

Table 3.4.1 Illustrative Presentation of Flexible Pavement Costs (Design System OPAC)

Design System, OPAC
Summary of the Best Design Strategies
in order of Increasing Total Cost

	1	2	3	4	5
Material Arrangement	ABC*	ABC	ABC	ABC	ABC
Intial Construction Cost	259,350	232,950	244,850	203,950	189,600
Overlay Construction Cost	27,300	49,300	52,750	70,450	81,300
User Cost	65,800	98,050	88,300	119,200	147,150
Routine Maintenance Cost	41,400	35,100	34,300	31,400	27,500
Salvage Value	-13,100	-13,550	-17,200	-15,100	-16,300
	------------	------------	------------	------------	------------
Total Cost	380,750	401,850	403,000	409,900	429,250
Layer Depth (millimetres)					
D(1) (Hot Mix Surface)	200	125	320	100	150
D(2) (Granular Base)	300	250		150	150
D(3) (Granular Subbase)		230		380	150
Performance Time (Years)					
(Resurfacing Time)					
T(1)	14.0	11.0	10.0	9.0	8.0
T(2)		22.0	21.0	18.0	15.0
T(3)				27.0	23.0
Padding & Level-Up					
+ Overlay					
0(1)	75	75	75	75	75
0(2)		75	75	75	75
0(3)				75	75

Notes: Design 1 - Deep Strength; 2 - Conventional; 3 - Full Depth; 4 - Conventional; 5 - Conventional
*ABC — number of letters indicates number of layers, i.e., 3 layers

D = depth; T = Time; O = Overlay

3.4.4 ADDITIONAL FACTORS IN SELECTING A DESIGN ALTERNATIVE

The economic evaluation of a pavement design alternative provides a basis for selection of the best alternative. However it is common for the alternatives to be essentially equal from an economics perspective. For most design situations, the following factors may affect the selection of the best economic alternative:

a. General performance and experience with similar pavements in the area
b. Adjacent, existing pavements
c. Geometric design considerations
d. Road system
e. Environmental Sustainability
f. Conservation of aggregates
g. Future potential for recycling
h. Competition considerations
i. Construction considerations
j. Local preferences and recognition of local industry considerations
k. Potential risks in availability of future overlay funds at planned times
l. Expected level of maintenance
m. Climatic considerations
n. Local soils and traffic characteristics
o. Safety considerations

Sound judgment is required in selecting a design alternative and economic evaluation is only one of the many factors to consider.

In the future, it is expected that alternatives will also be evaluated on the basis of their environmental sustainability.

3.5 LOW VOLUME ROAD DESIGN – UNDER 2000 AADT

Ministry Directives B-150 dated 82-07-29, and B-226 dated 89-01-03, divide the secondary highway system into three categories. It also sets design standards and guidelines for each category.

a. Major secondary highways comprise arterial highways serving important regional centres, communities, and resource areas. AADT volumes are generally greater than 1000 vehicles per day within a 10 years study period. Lower volume highways may be grouped with this category based on such factors as route length, lack of alternative routes and economic significance. The typical classification for this highway type is Rural Arterial Undivided (RAU).
b. Intermediate Secondary Highways are highways serving small towns and villages, recreational and resource areas. AADT volumes are generally 400 to 1000 vehicles per day within a 10 years study period. The typical classifications for this highway type are Rural Collector Undivided (RCU) and Rural Local Undivided (RLU).
c. Minor Secondary Highways are local highways providing access to recreational or resource areas or running parallel to a higher provincial facility. AADT volumes are generally less than 400 vehicles per day within a 10 years study period. The typical classification for this highway type is Rural Local Undivided (RLU).

According to the Geometric Design for Ontario Highways [2], the Functional classification system compared to other systems is illustrated in Table 3.5.1 below:

Table 3.5.1 Classification Systems Compared

Functional Classification System	Public Transportation and Highway Improvement Act	Provincial Highway Access Controls	Highway Inventory Management System
Freeway	Controlled Access Highway	Class I, II	Desirable King's Highway
Arterial	Controlled Access Highway King's Highway Secondary Highway	Class III, IV	Desirable King's Highway Desirable Secondary Highway
Collector	King's Highways Secondary Highways	Class V	Desirable King's Highway Desirable Secondary Highway
Local	Secondary Highway Tertiary Road Resource Road Development Road Roads in Territory without Municipal Organization remaining roads in this classification are part of the Municipal Road System	Class V	Desirable Secondary Highway Transfer Candidates

3.5.1 DESIGN FACTORS TO BE CONSIDERED

In general, the design factors considered in Section 3.3.3 also apply for low volume roads. In low volume roads, the aging due to environmental factors becomes more pronounced as the effect of traffic diminishes. The structural strength as expressed in GBE will be lower except where trucks predominate, but granular depths should be maintained to provide good drainage and frost protection. Truck percentages tend to be lower on low volume roads but may be significantly higher when the road serves a mineral or forest resource operation. In such cases it is imperative to get a truck count as well as the AADT. Table 3.3.3 provides structural design guidelines for the routine design method.

3.5.2 SURFACE TYPE, HOT MIX VS. SURFACE TREATMENT

Table 3.5.2 illustrates the secondary highway categories and their associated driving surface types. The designer, when selecting the appropriate driving surface type from Table 3.5.2 should consider the existing surface type for the overall highway with particular reference to adjacent projects. The driving surface construction may preclude the selection of an approved surface type for a particular category. Any change in category shall be referred to the Geotechnical Committee for approval.

Table 3.5.2 Secondary Highway Categories and Their Surface Types

Secondary Highway Categories	Driving Surface Type Design Options
MAJOR - Arterial highways serving regional economic centres, communities and resource areas. - Traffic volumes are generally greater than 1000 AADT. - Lower volume highways may be included based on route length, lack of alternative routes and economic significance. - Typical functional classification is rural arterial undivided.	Hot Mix
INTERMEDIATE - Collector highways serving small towns and or villages, recreational or resource areas. - Traffic volumes are generally 400-1000 AADT. - Typical functional classifications are: - Rural collector undivided - Rural local undivided	Surface Treatment,
MINOR - Local highways providing access to recreational or resource areas or paralleling a higher class provincial facility. - Traffic volumes are generally less than 400 AADT. - Typical functional classification is rural local undivided.	Gravel, Surface Treatment

The selection of the surface type for each category is dependent on roadway use, truck volume, time of construction and local concerns.

Northeastern Region completed a study in 2007 to help determine the most suitable surface types for low volume highways. Refer to reference [57] for more details.

3.6 CONSTRUCTION

Pavement performance and the service life of the pavement depend upon the quality of construction and materials. The specifications reflect many years of experience; therefore it is important to follow the specification in order to achieve the expected life of the pavement. This section attempts to address the question "What do we require in construction practices to ensure long-term performance?"

The ministry has moved from method specifications to end result (or performance) specifications. A one-year warranty is a typical requirement with the performance specifications to ensure the quality of the product. More recently, contract delivery methods such as minimum oversight (3 years warranty), warranty contracts (7 years), performance specification, and design-build contracts are being considered.

3.6.1 SEASONAL RESTRICTIONS

- Incorporation of frozen materials in the fill or the placements of materials on frozen ground are the most common problems in winter construction. When it is necessary to continue winter work, construction procedures should be carried out to ensure that the frozen materials are not incorporated into the work.
- The excavation procedures for rock and shale will be similar to the procedures used in the warm months. But it is important to note that shale must not be stockpiled at freezing conditions, and must immediately transported from cut or borrow sources and placed in the fill.
- To achieve the desired asphalt paving compaction, it must adhere to the temperature and/or date restrictions.

3.6.2 EQUIPMENT

- The equipment used should be capable of achieving the end result expected. In the case of compaction equipment, contractors must give consideration to the type and gradation of material, layer thickness and moisture content to ensure adequate compaction can be attained.

- The use of large vibratory compaction equipment adjacent to structures is restricted.
- The excavation of deep muskeg areas may require a dragline or long reach excavator.

3.6.3 PRODUCTION CONSTRAINTS

- Restrictions on the working hours are often applied in urban areas to minimize disturbance to surrounding residences, or to minimize traffic impact during peak periods.

3.6.4 PRODUCTION AND PLACEMENT

- Good drainage practices can often improve production. For example, ponding of water can be avoided by excavating uphill and on embankment construction; maintaining a crown to shed the water and avoid the build-up of granular windrows along the shoulders, etc...
- Wet soils must undergo a reduction in moisture content to achieve optimum densities. A drying procedure such as discing or reworking can be effective in some situations.

3.6.5 PREPARATION OF FOUNDATIONS

- The stability and settlement characteristics of foundation soils for embankments and culverts must be determined and incorporated into designs.
- Specific construction procedures and instrumentations are often required to avoid failures and to achieve an acceptable long-term pavement performance by minimizing post construction settlement.

3.6.6 COMPACTION

- Adequate compaction must be obtained throughout the pavement structure layers, embankments and subgrades to minimize settlement and to increase the bearing strength of the pavement.
- The ministry uses an End Result Specification (ERS) to determine the acceptability of compaction [58].

3.6.7 QUALITY CONTROL

- Quality Assurance (QA) – planned actions necessary to provide adequate confidence that a product or service will satisfy given requirements for quality. Quality assurance is the process used to ensure that quality controls and service aspects are carried out, and to monitor and record the appropriate verifications applicable to activities during each phase of a project.
- Quality Control (QC) – operational techniques and activities aimed both at monitoring a process and at eliminating causes of unsatisfactory performance. QC refers to technical detail and is the method manufacturers and builders used to ensure their materials are up to standard and products are fabricated or built correctly.
- Statistical methods are used for acceptance of material and work for concrete, hot mix, granular materials and compaction to ensure consistency and quality.
- Quality assurance procedures are covered in the appropriate specifications, directives and special provisions.

3.6.8 PAVING

- Adequate paving must be obtained to minimize paving joints, such as end load segregation and longitudinal joints with adjacent lanes. Care should be used when compacting unconfined joints.
- Use echelon paving to minimize longitudinal joints.

3.6.9 DETOURS

- Detours within the right-of-way or along adjacent roads permit good construction practices while ensuring orderly traffic flows. Generally, the anticipated life of detour dictates its design.
- If municipal roads are to be used as detours, condition surveys and soils investigations should be undertaken to assess the structural adequacy of the pavement because the ministry may need to restore the road to the original conditions according to the agreement with the municipality. In the case of weak pavements and structures, it may be necessary to strengthen them prior to their use as detours.

3.6.10 TRAFFIC CONTROL

- Special precautions should be taken to ensure that construction equipment can be operated safely without making it hazardous to passing traffic [59].
- To maintain traffic over gap graded subbase or pulverized bituminous pavement materials, a special provision may require the contractor to conduct his operation within a specified distance to minimize exposure to traffic ahead of the base course or paving operations.

3.6.11 STAGING

- Stage construction is often a required procedure to be used under specific circumstances such as partial cut and fill construction to allow for the movement of traffic. Another instance where stage construction is used to pre-load compressible soils to minimize long-term settlements. It can also be used to limit the loads on soft ground so that the soil can gain strength with time before applying the full fill height.
- It is important for the construction staff to be aware of the purposes of the stages so that adjustments can be made to avoid risk of failures.
- In areas where stage construction is critical, the staging should be indicated on the contract drawings and special provisions.

3.6.12 GEOGRAPHIC CONSIDERATIONS

Due to its location and glacial history, Ontario has a variety of topographic features, soil conditions and temperatures. These factors can create design and construction problems. Some of the geographic areas affected are:

Ottawa Valley

The sensitive marine clay below the desiccated upper layer is unsuitable for fill material unless special treatment is used.

Precambrian Shield Area

Deposits of deep soft clay are often found in the valleys between rock outcrops. Special design techniques and considerations are required to build over these areas to avoid differential pavement performance, excessive settlements, or embankment failures. The bedrock surface can vary dramatically in elevation over very short distances. Particular attention should be given to the determination of the bedrock profile. Refer to reference [60] for more details on establishing rock elevation.

Southwestern Ontario - (Chatham, Windsor Area)

The topography is flat and, while the parent material is suitable for embankment purposes. The only way to obtain earth borrows and ensures adequate drainage is by the construction of water collection ponds and by wide and deep drainage ditches.

Northern Ontario

In the clay belt areas, much of the terrain consists of varved clays and swamps. The main problem with the clay is high moisture content. The northern Ontario construction period is shorter than that of southern Ontario because of climatic conditions. The frost does not usually leave the ground until mid-summer.

Niagara Escarpment Area

Procedures for the opening of wayside pits and quarries in Ontario are screened by provincial legislation which is administered by the Ministry of Natural Resources. Pits and quarries located within the Niagara Escarpment area require special attention. Directive PHY-B-102 outlines the procedures for handling wayside pits and quarries in MTO projects.

3.7 DETERMINING THE SCOPE OF SOILS INVESTIGATIONS

The complexity of a pavement construction project should be properly reflected in the scope of a soils investigation. Soil investigations are required to determine the soil types and soil and bedrock conditions that may influence the design and performance of the pavement. For the purposes of this document, a soils investigation refers also to borings and cores of the pavement structure.

Investigations of pavements, soil and rock are carried out using one or more of the following: hand augers, power augers, peat samplers, test pits, core drills and/or geophysical equipment as detailed in Section 1.1. Generally, if access is unrestricted, borings are advanced using power auger and disturbed samples are collected for laboratory testing purposes. High fills, deep cuts, and areas of soft soil conditions require a more rigorous soils investigation in order to facilitate appropriate slope stability or settlement analyses. For these and other challenging site and soil conditions, the Foundations Group of the Pavements and Foundations Section should be consulted.

Prior to commencing a soils investigation it is important to become familiar with the geology, physiography and general soil types in the area. A field visit should be made to assess overall site conditions and to identify problem areas and extent for more detailed investigation. Accessibility of the site for equipment, the presence of overhead and buried utilities and traffic control requirements should also be assessed. This information will assist in planning and conducting the investigation. Refer to the Provincial Pavement Engineering Investigation Guidelines (November 2009) and Reference [61] for more detail.

Pavement construction projects may range from single lift overlays to full reconstruction of an existing pavement to construction of a new facility. Guidance in developing the scope of a soils investigation for different project requirements is provided in the following subsections.

3.7.1 RESURFACING PROJECTS

Borings, typically 1.0 to 1.5m deep, are placed along the pavement edge and within the roadway shoulder to ascertain the existing pavement structure. Borings are also advanced where structural problems are evident within the pavement surface. Problem areas such as frost heaves require sufficient investigation to isolate the problem area. Generally, borings are advanced at 25 m intervals on either side of the frost heave until a uniform soil is encountered. To establish existing hot mix depths, cores are taken according to MTO Directive C-145 “Asphaltic Concrete Pavement Coring Procedures”.

3.7.2 WIDENING

For proposed roadway widening to accommodate speed change lanes and tapers, the borings should determine if the existing base and subbase granular courses are full width, wet or contaminated and if the subgrade soil is the same as that of the driving lane. This information is needed to decide whether the existing shoulder should be excavated and whether top soil is present. For projects with minimal pavement widening of 0.3 to 0.6 m on both sides, the frequency of spacing of borings is similar to that for resurfacing.

3.7.3 NEW CONSTRUCTION OR RECONSTRUCTION

For earth cuts on a new construction alignment, borings are placed at 20 m spacing along centre-line to a depth of at least 1.5 m below profile grade or to refusal on bedrock or boulders.

Where side hills exist, additional borings are placed on the uphill side. Where there is an existing roadway, borings are usually placed at the edge of the travelled surface nearest the proposed centre line. If the earth cut material is considered suitable for fill, drilling holes a few metres deeper is desirable in case the grade line is lowered to utilize more of the material. If bedrock is encountered less than 1.2 m below the proposed profile grade, the boring spacing is decreased to a maximum of 15 m along centreline and additional borings are offset in the vicinity of the proposed ditch line on both sides at 30 m intervals. Where existing and proposed horizontal alignments may coincide, borings are placed at both pavement edges in proposed rock cut sections.

In proposed fill areas, boreholes are advanced at intervals of 50 to 100 m and to a depth equivalent to the proposed height of fill or 1.5 m, whichever is greater. Borings are always placed at proposed grade points for purposes of arriving at transition treatments.

3.7.4 REALIGNMENT

Horizontal curves are often "flattened". Boreholes are placed along the revised centreline, a maximum of 50 m apart. Additional borings should be placed where the new and old alignments meet for proper design of transition treatments.

3.7.5 MUSKEG

Where muskeg is present on the existing roadway, boreholes are advanced using power equipment at the edge of the pavement or on the shoulder. The borings should be taken to firm inorganic bottom. If the equipment is not capable of reaching firm bottom, a hand auger or a peat sampler can be used and pushed to firm bottom. Additional test holes are placed at a distance of 25 to 50 m right and left of centreline. The change from organic to soft clay should always be identified where soft clay underlies the organic material. On new horizontal alignment through swamps, soundings are placed along centreline at 25 m intervals. In addition, any sudden changes in muskeg depths should have additional soundings to establish the limits of the organic deposit.

Occasionally, the bearing capacity of the organic root mat is stronger than the underlying strata due to the interweaving of the root system. The field vane test may be used to determine the strength of the organic and clay material at various depths ensuring that decayed wood or roots do not create false readings. It is essential that at least some soil borings are advanced to a sufficient depth to ensure firm bottom.

3.7.6 SEWERS

Boreholes should be placed along the exact alignment of the proposed sewer. This is especially important if shallow bedrock is encountered. All soil borings should be advanced to at least 0.5 m below (and deeper if site conditions warrant) the invert of the pipe or manhole unless bedrock is encountered above that elevation. Borings are typically spaced at 25 to 50 m intervals. If bedrock is encountered, the spacing should be reduced to 10 to 15 m intervals (maximum).

3.7.7 CULVERTS

Along a new horizontal alignment, boreholes should be advanced along centreline at all obvious culvert locations and at the ends of the culvert if a change in soil conditions is anticipated. The boreholes should be of sufficient depth to indicate what material the culvert will be founded on and whether settlement, and its extent, can be anticipated. When an existing culvert is to be replaced or extended, a borehole should be placed at either end with the hand auger to determine the soil type and the depth to firm bottom.

If frost heaving or settlement has occurred at an existing culvert, the backfill should be investigated. Borings should be advanced adjacent to the structure or pipe for a sufficient distance to establish the grading and frost-susceptibility of the existing materials and/or granular tapers.

3.7.8 SAMPLING AND TESTING REQUIREMENTS

The number of samples taken during the soils investigation depends upon the complexity of the job. This information is covered in Section 1.1.3.

3.8 SOIL PROFILE

In a geological sense, a soils profile is commonly known as a "vertical section of a soil showing the nature and sequence of the various layers as developed by deposition or weathering or both" [62]. In the context of the pavement design report, the soils profile is a longitudinal profile of the topography taken along centreline of the highway showing a gradeline proposed by the Planning and Design Section upon which the soils data is placed.

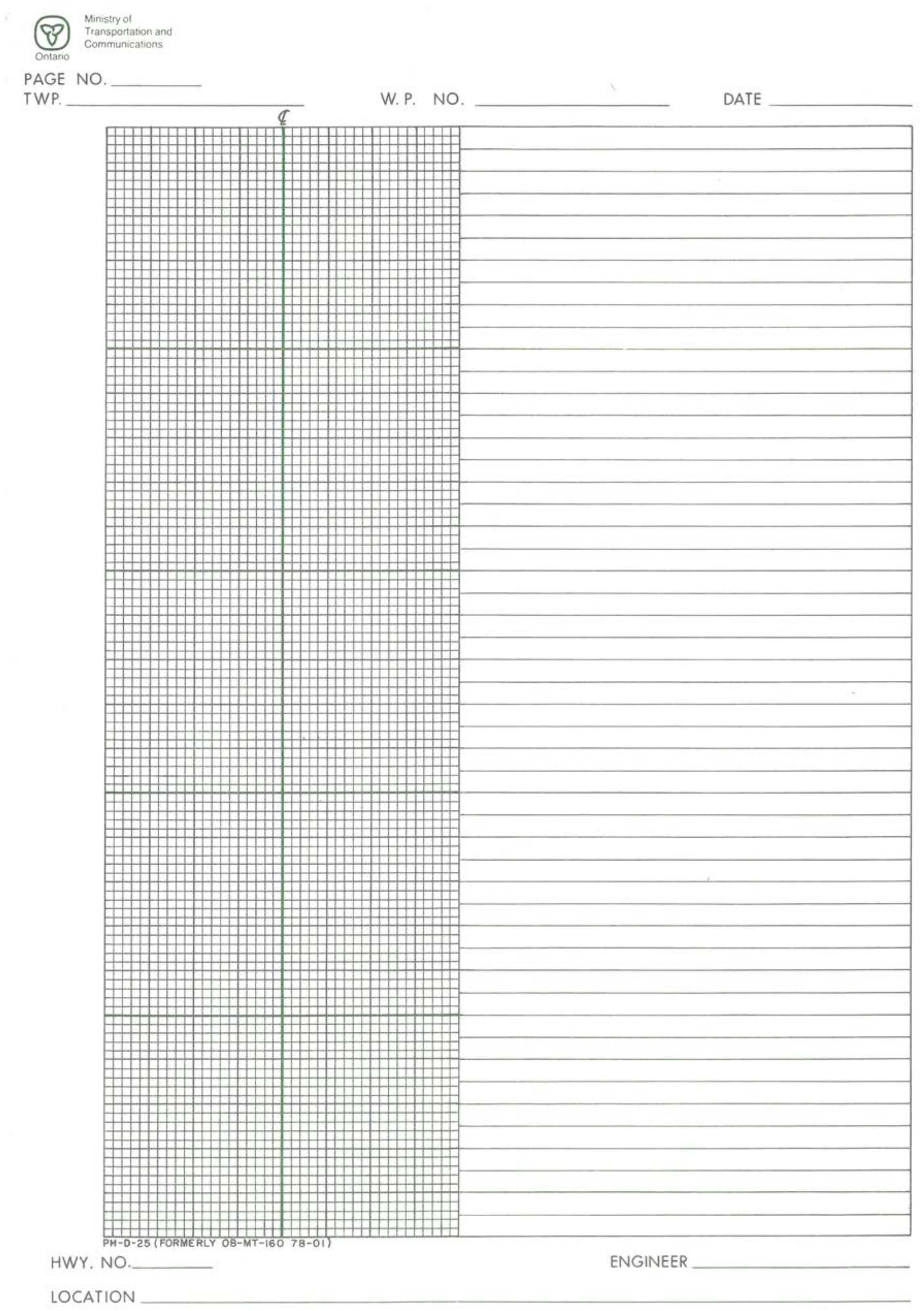
All of the pertinent information obtained from the soils investigation such as borehole logs, bedrock and muskeg profiles, soft layers, vane shear strengths, water table and boulders, along with any laboratory test results are plotted on the soils profile. On existing roads, information regarding pavement performance is also shown.

Pedological surveys are carried out primarily on new alignments to obtain the boundaries of the various soil profiles and soil types, bedrock outcrops, muskeg locations and any other features which would be of use for pavement design purposes. This information is recorded and sketched on Form PH-D-25 (Figures 3.8.1 and 3.8.2). Sketching is also done at frost heave locations and pavement distortion areas. All of the information obtained from the pedological surveys and the sketches along with the laboratory test results is placed on the soils profile (Figure 3.8.3).

All of the borehole data, test results and performance information are analyzed and summarized along the base of the soils profile with tentative recommendations required for design and construction purposes.

The soils profile is prepared in conjunction with the Pavement Design Report which describes recommendations in more detail. These recommendations may include such things as type and suitability of cut materials for embankments, subgrade soils, subexcavations, granular thicknesses, muskeg treatments, drainage requirements, embankment stability, grade revisions and culvert treatments. Where a soils profile is included in the contract documents, only factual information is presented. Any interpretative information which could influence the bid prices should be avoided. Interpretative information may be included on the soils profile submitted to the Regional Planning and Design Section for the preparation of contract documents.

Where a soils profile is not prepared, a summary of borehole logs, core logs and test results are included with the contract documents.


Ministry of
Transportation and
Communications
Ontario

PAGE NO. ____________

TWP. ____________ W. P. NO. ____________ DATE ____________

℄

PH-D-25 (FORMERLY OB-MT-160 78-01)

HWY. NO. ____________ ENGINEER ____________

LOCATION ____________

Figure 3.8.1 Pedalogical Survey Form

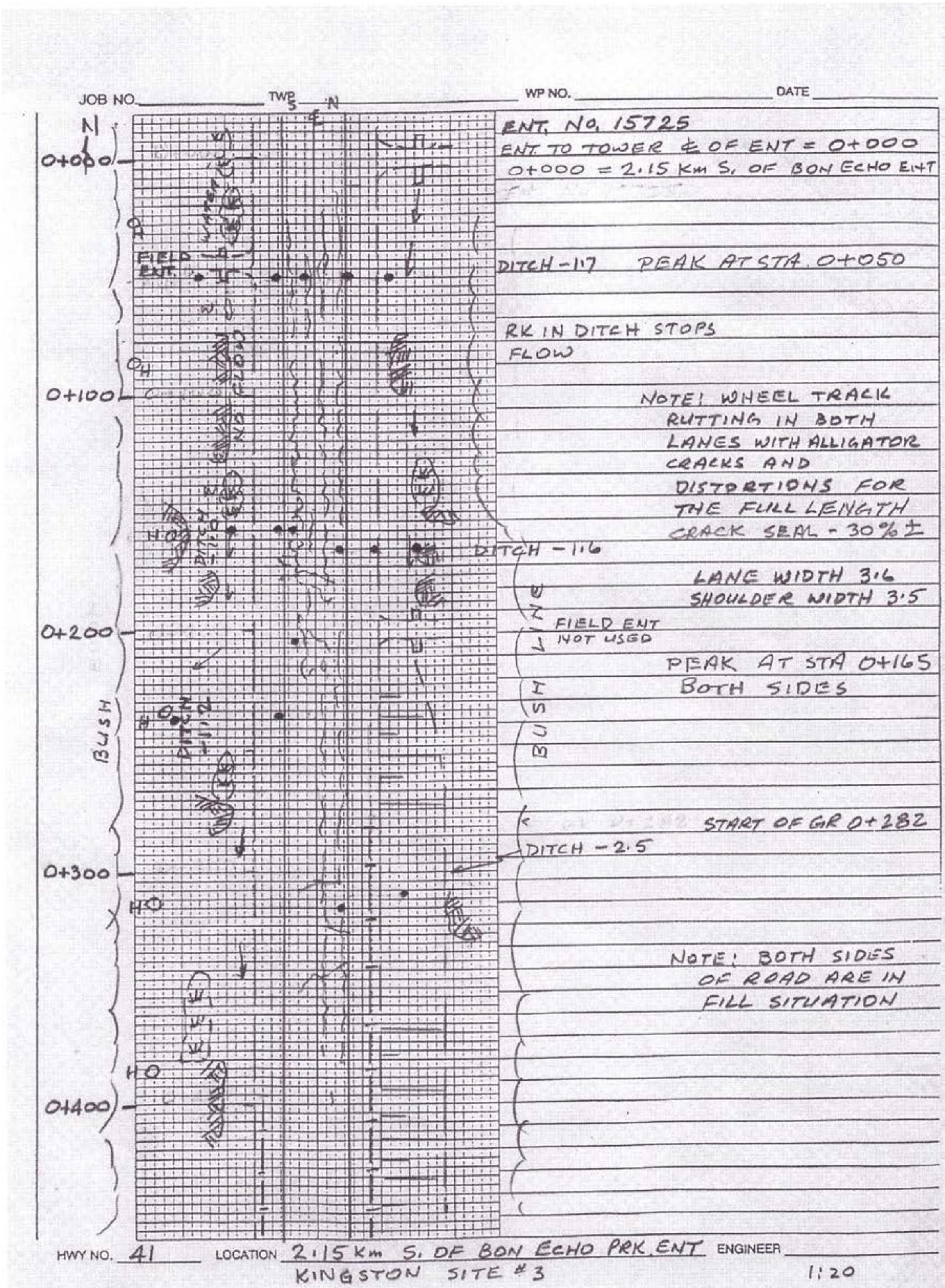


Figure 3.8.2 Pedalogical Survey Example

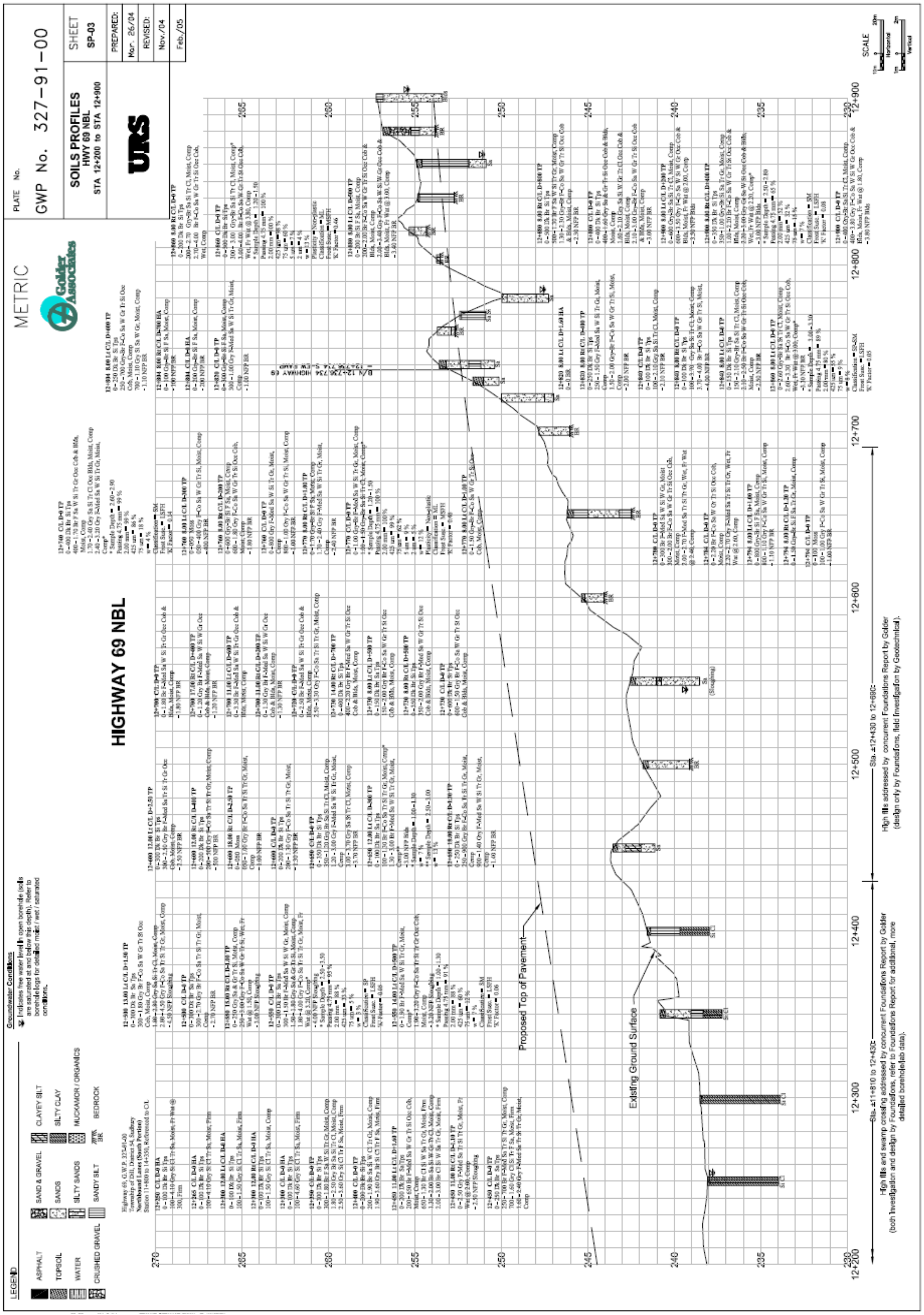


Figure 3.8.3 Soils Profile

3.9 PAVEMENT DESIGN REPORT

All of the data gathered for a project, from various performance, soils and granular surveys, laboratory test results, field observations to knowledge of problems that had previously been encountered in the area, are analyzed and presented in the form of a Soils Profile and/or a Pavement Design Report. This information is used by the Regional Planning and Design staff in preparing contract tendering documents and by the Contracts and Operations staff to identify potential problems during the construction stage.

The Pavement Design Report should contain all of the soil information within the right-of-way needed for the design and construction of the project as well as the soil information in the vicinity of the project for assessment of earth borrow and granular materials and their suitability as construction materials. The Pavement Design Report should include limitations on the use of excavated material and the use of appropriate construction equipment, and all recommendations regarding pavement structure depths, transition point treatments, muskeg and soft ground treatments, type of pavement and any other points deemed pertinent in arriving at the most economical and structurally adequate design.

The exact content of a Pavement Design Report will depend upon the site-specific character of the project. The overall format however has been standardized for consistency so that certain kinds of information will be included regardless of the nature of the project [61], [62], [63]. Typical headings and descriptions that make up a Pavement Design Report are shown in Appendix A.

3.10 SOILS AND PAVEMENT DATA IN CONTRACT DOCUMENTS

The following is an overview of the soils and pavement related information to be provided as part of the contract documents:

- All borehole logs and laboratory test data are presented on Borehole Data and Core Log Data sheets within the contract drawings
- When available, the soil profile is provided as part of the contract information package.
- In 'no plans' format contracts, the soils and pavement data is presented in the contract special provisions
- The recommended pavement designs are shown on Typical Sections as part of the contract drawings.

REFERENCES TO CHAPTER 3

[1] Transportation Association of Canada, Pavement Design and Management Guide, TAC, Ottawa Canada, 1997.

[2] *Geometric Design Standards for Ontario Highways,* 1994.

[3] *Scope and Cost Report Guideline*. Design and Contracts Standard Office. February 2010.

[4] DD 76-62, OD 76-78, *Criteria for the Design of Pavement Structures.*

[5] *Construction Manual -Bituminous Pavement Construction,* P6-6 to P69.

[6] *Construction Manual - Soils Testing and Construction Practices, W-76.*

[7] Directive PHM-C-001, *Surface Course Hot Mix Types. Freeways and other King's Highways*

[8] *Canadian Highway Bridge Design Code* (CHBDC), 2006.

[9] *Truck Weights and Bridge Design Loads in Canada*, SRR-79-12, 1978

[10] Hajek, J., Smith, K.L, Rao, S.P., and Darter, M.I., *Adaptation and Verification of AASHTO Pavement Design Guide for Ontario Conditions,* Ministry of Transportation Material Publication, MI-183, Revised March 2008.

[11] *Life Cycle Costing 2006 Updates*, Applied Research Associates, August 23, 2007.

[12] Tighe, S., *Using RWIS to Improve Seasonal Load Restriction Policies*, Ministry of Transportation, 2005 to 2007.

[13] *Pavement Structure Design Guidelines*, MTC, June 1973.

[14] MTO Directive C-17, *Frost Heave Treatments using Expanded Polystyrene Insulation*, 78-11-23.

[15] Chisolm, R. A. and Phang, W. A., "*Aspects of prolonged exposure of pavements to sub-zero temperatures*", Report RR225 - Parts 1 and 2, Province of Ontario Ministry of Transportation and Communications, Policy Planning and Research Division, Downsview, Ont., Canada, December 1981.

[16] *Construction Manual - Soil Testing and Construction Practices.*
a) Frost Action, P7-8 1 to P7-84.
b) Transition Point Treatment, P7-85 to P7-88.
c) Backfill Practices - Structures, Culverts and Sewers, P7-89 to P7-99.

[17] R. Chisholm, *Frost Depth Penetrations*, Ontario MTC 1970-74.

[18] *MTO Soils Manual*, January 1980, Chapter 15, page 15-1 to 15-3.

[19] J.B. MacMaster and G.A. Wrong, *The Role of Expanded Polystyrene in Ontario's Provincial Transportation System*, 1986.

[20] G.J. Chong, W.A. Phang and G.A. Wrong, *Manual for Condition Rating of Flexible Pavements -Distress Manifestations*, MTC**,** August 1975.

[21] G.J. Chong, W.A. Phang and G.A. Wrong, *Manual for Condition Rating of Rigid Pavements -Distress Manifestations*, MTC, April 1982.

[22] Lane. B. and Kazmierowski, T., *Guidelines for the use of Life Cycle Costing*

Analysis on MTO Freeway Projects, Ontario Ministry of Transportation, MERO-018, March 2005.

[23] Directive B-82, *Pavement Selection*, 80-04-23.

[24] E. J. Yoder, M. W. Witczak, *Principles of Pavement Design*. 1975.

[25] Ryell J., Corkill J.T., *Long-term Performance of an Experimental Composite Pavement*, Ontario MTC, Research Report No. 184, 1972.

[26] Roads and Transportation Association of Canada, *Pavement Management Guide,* 1977.

[27] Jung F.W., Kher R., Phang W.A., *A Performance Prediction Subsystem - Flexible Pavement, Ontario Ministry of Transportation and Communications, 1975*. Report No. RR-200

[28] Phang W.A., *Flexible Pavement Design in Ontario,* Transportation Research Board, Washington, D.C., Record No. 5 12.

[29] Jung F.W., Phang W.A., *Elastic Layer Analysis Related to Performance in Flexible - Pavement Design*, Ontario Ministry of Transportation and Communications Research Report No. 191, 1975.

[30] MTO Bituminous Section, *MTO SuperPave and SMA Guide*, MERO, March 2008.

[31] American Association of State Highway and Transportation Officials, *AASHTO Guide for Design of Pavement Structures,* 1986.

[32] American Association of State Highway and Transportation Officials, *AASHTO*

Guide for Design of Pavement Structures, 1993.

[33] Phang W.A., *Four Years' Experience at the Brampton Road Test*, Ontario MTC, Research Report No. 153, 1969.

[34] Haas R.C.G., Camel N.L, Morris J., *Brampton Road Test: An Application of Layer Analysis to Pavement Design*, Ontario MTC, Research Report No. 182, 1972.

[35] *Guide For Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure, Part 1 – Introduction*, NCHRP, March 2004.

[36] Washington State Department of Transportation, *WSDOT Pavement Guide* **http://training.ce.washington.edu/WSDOT/**

[37] *Guide For Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure, Part 3 – Design Analysis – Chapter 3*, NCHRP, March 2004.

[38] The Asphalt Institute, *Thickness Design -Asphalt Pavements for Highways and Streets*, Manual Series No. 1, 1981.

[39] Kazmierowski T.J., Wrong J.A., Phang W.A., *Design, Construction and Performance of Four Experimental Concrete Pavement Sections in Ontario,* Ontario MTC, 1985.

[40] Canadian Portland Cement Association, *Thickness Design for Concrete Highway and Street Pavements*, 1984.

[41] American Concrete Pavement Association, "*Pavement Analysis Software*", (PAS), ACPA, 1993.

[42] Canadian Portland Cement Association, "*Thickness Design for Concrete Highway*

and Street Pavements", EB209, Toronto, 1984.

[43] American Concrete Pavement Association, "*Design of Concrete Pavement for City Streets*", ACPA, Arlington, Heights, 1992.

[44] Portland Cement Association, "*Design of Concrete Pavement for city Street*", IS 184, Skokie, 1992.

[45] Darter, M.I., "*Structural Design for Heavily Trafficked Plain Jointed Concrete Based on Serviceability Performance*", Transportation Research Record 671, TRB, 1978.

[46] *Guide For Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure, Part 3 – Design Analysis - Chapter 4*, NCHRP, March 2004.

[47] Guell D.L., *Comparison of Rigid Pavement Design Methods,* American Society of Civil Engineers Proceedings, 1985.

[48] Chong G.J., Stott G.M., *Evaluation of the Dynaflect and Pavement Design Procedures*, Ontario DTC Report, Report No. IR-42, 1971.

[49] The Asphalt Institute, *Asphalt Overlays for Highway and Street Rehabilitation*, Manual Series No. 17, 1983.

[50] *Guide For Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure, Part 3 – Design Analysis - Chapter 6*, NCHRP, March 2004.

[51] American Concrete Pavement Association, "*Guidelines for Concrete Overlays of Exiting Asphalt Pavements*', TB-009P, ACPA, 1991.

[52] Portland Cement Association, *Guide to Concrete Resurfacing Designs and Selection*

Criteria, 1981.

[53] *Guide For Mechanistic-Empirical Design of New and Rehabilitated Pavement Structure, Part 3 – Design Analysis - Chapter 7,* NCHRP, March 2004.

[54] Kher R., Phang W.A., *Economic Analysis Elements*, Ontario MTC, Research Report No. RR-20 1, 1975.

[55] Winfrey R., *Economic Analysis for Highways*, International Textbook Co., Scranton, PA,1969.

[56] Haas R.C.G., Hudson W.R., *Pavement Management Systems*, McGraw-Hill Inc., 1978.

[57] *Surface Type for Low Volume Roads, Assignment No. 5004-E-0050 Final Study Report*. Applied Research Associates. March 12, 2007.

[58] *Construction Specification for Compaction*, OPSS 501.

[59] *Ontario Traffic Manual, Book* ., Temporary Conditions, March 2001.

[60] Robert G. Gorman, Chris A. Rogers, *Recommended Practice For Establishing Rock Elevation For New Highway Construction,* MERO report #30, January 2007.

[61] *Provincial Pavement Engineering Investigation Guidelines Version 1.0*, Ontario Ministry of Transportation. November 2009.

[62] *MTO Soils Manual*. January 1980.

[63] Construction Manual, Part 7.

APPENDIX A:

TYPICAL PAVEMENT DESIGN REPORT FORMAT

PAVEMENT DESIGN REPORT

Highway 3	Johnstown to Torrence		15.0 km
Work Project 88-560	LHR/OS	From	To

Profile No. C-1342 (Soil Profile 3-A-41)

Design Summary

Synopsis - Separate Page

General Data

- General statement of project relative to overall highway network and the significance of the project
- Program year
- Existing roadway conditions, pavement width, shoulders, etc.
- Limits of projects
- Summary of proposed work
- Last reconstruction or resurfacing
- Maintenance history
- Patrol yard development, if included in project

Design Criteria

- Justification, accidents, present and projected traffic volumes and commercial vehicle types
- Design proposals, alignment and grade, cross-section width, shoulders, slopes, grades, intersections, curbs, barriers, signals

Pavement Performance (Existing Conditions)

- Year built and/or resurfaced; pavement condition such as main distresses, riding comfort, roughness, drainage concerns, cross-fall, superelevation, shoulder cross-fall and width
- Pavement performance at: frost heaves, grade points, structures, culverts, ditches, sewers, slope failures; pavement performance of adjacent projects

Physiography and Soils Data

- Physiographic Region (Putnam and Chapman), geological earth and rock history
- Land form, land use, soil erodibility
- Frost penetration depth for the area
- Degree days freezing index
- Bedrock hardness, e.g., igneous, shale, etc.

Investigation

- Scope, limits and time of investigations, equipment used, laboratory testing
- Investigation of potential S.S.M. sites, isolating limits of unsuitable soil types and the limits of bedrock and organic deposits
- Determine causes of frost heaves. Investigate aggregate hardness (recycling)
- Pavement cores to determine depths, gradation asphalt cement content and penetration Foundation investigations
- Skid resistance, pavement roughness and deflection investigations, etc.

Attachments to Pavement Design Report:

- Log of Borings and Laboratory Test Results
- Bituminous Pavement Core Data and Analysis
- Soils Survey Data statement

Borrow Materials

- The location, type, suitability, quantity, moisture conditions of earth borrow and select subgrade materials
- Zoning restrictions, environmental concerns
- Lightweight fill materials

Granular Materials

- Type, quantity, suitability, location of granular type materials
- Proximity to project
- Structure and sewer backfill
- Sewer and culvert bedding
- Environmental restrictions
- A.S.L. and/or Concrete Designated Source List

Alignment and Gradeline

- Problems and reasons for alignment and grade changes such as minimize loading on swamps
- Avoid shallow cuts or bouldery till
- Utilize existing granular in cuts, additional borrow
- Minimize transition treatments

Alternative Pavement Designs Considered

- Design approaches considered.
- Special conditions and treatments
- Previous designs - MTO method, DARWin, AASHTOWare Pavement ME Design and OPAC 2000
- Life cycle cost estimates and benefits
- Alternative pavement types considered
- Recommended design and justification
- Subgrade improvement requirements

Recommendations and Construction Features

- Pavement types and depth
- Recycled hot mix
- Granular types and depth
- Conversion factors
- Widening
- Resurfacing
- Levelling
- Milling
- Padding
- Reclamation

Pavement Types and Depths-Binder and Surface Course

- Mix type
- Total depth
- Shoulders - paved - depth, type
- Modified mixes

Recycled Hot Mix

- Reclaimed Asphalt Pavement (RAP)
- Ratio RAP/Virgin
- In-place recycling opportunities
- Aggregate type / properties
- Waste allowance

Granular Types and Depths

- Width
- Stabilized, modified, drainage layers
- Shoulders

Conversion Factors

- Applied to Granulars
- Lightweight aggregates
- Volume adjustment factors
- S.S.M. factor (Estimating Manual gives range)

Cut Materials and Design

- Length, width, depth, benching, staging, drainage, ditches
- Modification to slope treatment
- Use of geosynthetics, bulking factor

Treatment or Disposal of Cut Materials

- Selection, identification, use, moisture conditions, suitability
- Location, limits, quantities
- Disposal in or outside right of way
- Drying
- Seasonal problems

Transition Point Treatments

- Recommended Depth of "t" and "H". ("t" for depths of transition treatment "H" depth of "A" and "B" horizons).
- Show depths of earth and rock treatment (Standards OPSD 205.1 and 205.05).

Excavations or Frost Heave Treatments

- Locations of frost heaves
- Frost-susceptible soils depths
- Replacement or polystyrene
- Drainage
- Paved shoulders

Bouldery Subgrade or Surface Soils

- Location and extent shown on profile
- Size of boulders
- Treatment standard
- Special provisions
- Cut section
- Disposal
- Placement and compaction
- Isolated rock knobs and earth pockets
- Quantity of boulders in excess of 1 m^3

Slope Treatment

- Need for soil treatment; seeding; sodding; staking; granular or stone blanket; drains; benching; riprap; planting; angle of repose of subsoil
- Need for geotextiles
- "K" factors

Embankment Materials and Design

- New construction
- Fill widening
- Widening granular lifts
- Service road embankment or gravel road designs
- Entrances
- Detours
- Shoulders
- Side hill cut material
- Lightweight fills surcharges and benching
- Reference to Foundation Design Report, where applicable

Embankment Materials

- Uniform subgrades
- Frost susceptible material below frost zone
- Fine grain soils problems
- Flatten slopes
- Improve surface drainage
- Early vegetation

Muskeg Treatment

- Summary of deposits, widths and depths, nature and texture, wet dry compressible, recommended treatments, excavation standards, drainage
- Rate of excavation, percent displacement when partial excavation is used, reference to foundation design treatment

Drainage

Culverts –

- Locations and types (concrete, steel, plastic)
- Contractor option and manufacturer's requirements
- Soil types at foundation (rock, wet, dry, open or closed footings, erosion likely)
- Camber on all steel culverts; structure backfill (compaction; permeability) bedding
- Standard or revision to poly-gaskets required

Sewers and Sewer Excavation –

- Sewer excavation soil known or not
- Bedding class
- Backfill material (granular or native)
- Depths to rock, if uncertain recommend unit price items for earth and rock

excavation, if rock depth known use lineal measure

- Replacing existing pavement at sewer or utility cuts, pavement restoration, depths - hot mix, granular backfill or unshrinkable fill.

Stream Diversion –

- Location, length, use of excavated material
- Fills for old bed
- Rip-rap, Gabions, Weirs
- Contamination

Erosion Control - Erodibility of soils, treatments, "K" factors.

Ditches

- Modified drainage design (ditch depth)
- Geotextiles, rock profile in ditch
- Erosion, sequence of construction
- Plastic drains, stone drains

Subdrains or French Drains

Location, type, size, function, outlets, backfill, rock excavation, plastic perforated pipe, geotextile fabrics required.

Other Design Features

Stripping

- Width of stripping
- Extent and depths expected
- Quantity
- Maintenance needs
- Stockpile

Pavement Removal

- Pavement type, depth, limits, disposal and treatment
- Quantity
- Pulverization, crushing, milling, cold planing

Detours

- Staging
- Stripping
- Method of construction
- Materials
- Removal, Disposal

Utilities

- Staging
- Trench backfill
- Drainage

Modifications to Specifications - Special Provisions

- Permissible variations to use local materials
- Limiting factors and effects
- Preloading
- Stage construction
- Instrumentation
- Geotextiles

Addenda

The Pavement Design Report is based on information available at the time of writing. When new data become available, changes are made to the regional design recommendations and issued in a technical memo.

Affected offices should be aware of subsequent memoranda issued as addenda to this report.

(Blank Page)

Chapter 4:

MAINTENANCE, PRESERVATION AND REHABILITATION

(Blank Page)

4 Maintenance, Preservation and Rehabilitation

4.1 MAINTENANCE, PRESERVATION AND REHABILITATION STRATEGIES

The ministry procedures for maintenance, preservation and rehabilitation of flexible and rigid pavements have been largely covered in the Maintenance Manual: Maintenance Quality Standards (MQS) and Maintenance Best Practices (MBP) [1], and Pavement Maintenance Guidelines: Distresses, Maintenance Alternatives, Performance Standards, SP-001 [2]. These manuals provide the contractor with specific guidance for the identification of pavement distresses and the procedures, materials and manpower/equipment requirements for completing the necessary repairs of these distresses.

The identification of pavement problems is largely accomplished through an ongoing evaluation of the Pavement Condition Index (PCI), formerly described as Pavement Condition Rating (PCR) [3]. These inspections provide a continuous record of the condition of a given pavement section. When reviewed with the Highway Maintenance Needs Reports, a complete summary of the pavement condition is provided. This indicates the rate of pavement deterioration, the type(s) of pavement distress and their causes (load, environment, age, materials, etc.), previous maintenance actions that have been employed and their impact on the pavement condition, etc. This information is used to select and evaluate maintenance alternatives for the section of concern, and to identify the most practical and cost-effective rehabilitation strategy.

Pavement improvement strategies can be divided into the following [4]. Figure 4.1.1 is a graphical illustration of the various pavement improvement strategies.

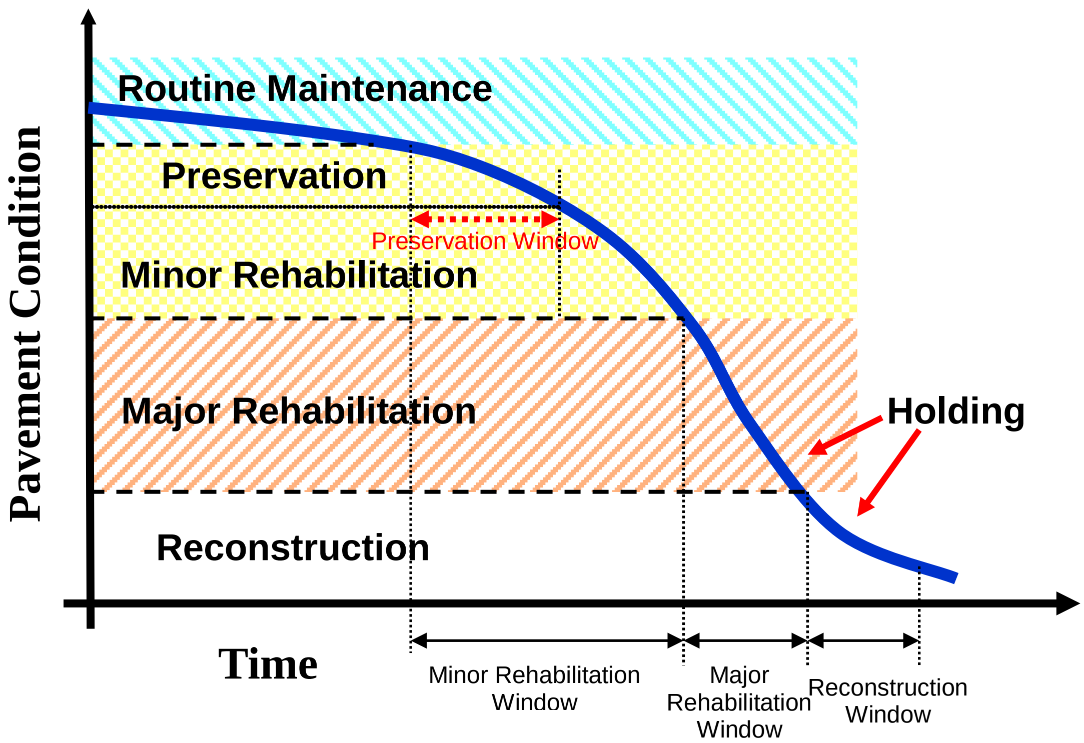


Figure 4.1.1: Pavement Improvement Strategies and Related Schedule

Routine Maintenance – reactive, timed and executed activities employed to ensure pavement life until deterioration of the pavement reached a minimum acceptable level of serviceability that is more cost effective to rehabilitate the pavement. Usually the pavement distress and frequency require localized repairs that can generally be carried out under regional maintenance contracts.

Rehabilitation – involves the application of appropriate measures, including reconstruction, to extend the life of an existing pavement structure when roughness, lack of structural adequacy, or excessive surface distress results in an unacceptable pavement.

> ***Minor* -** consist of non-structural enhancements made to existing pavement sections to extend its service life without involving the granular layers. Because of the non-structural nature of minor rehabilitation techniques, these types of treatments do not significantly improve the load carrying capacity of pavements.

***Major* -** consist of structural enhancements that extend the service life of an existing pavement and/or improve its load carrying capacity. Major rehabilitation treatments include restoration treatments and structural overlays without involving the subbase and/or subgrade layers.

Reconstruction - consist of replacing the entire existing pavement structure at the end of the anticipated service life (typically after 2 or 3 rehabilitation cycles) which is either failed or has become functionally obsolete. Reconstruction usually requires the major removal and replacement of the existing pavement structure, including granular layers and/or subgrade.

Preservation - proactive, consisting of well timed and executed activities to prevent premature distresses and to slow the deterioration rate of a structurally sound pavements having significant remaining service life. Preservation treatments typically include minor (non-structural) rehabilitation treatments.

Holding - strategy that prolongs the life of pavement to maintain acceptable levels of functionality or safety until rehabilitation or reconstruction can be completed.

The strategies and corresponding flexible and rigid pavement maintenance, preservation and rehabilitation activities are illustrated in Table 4.1.1 below.

Table 4.1.1 Pavement Maintenance, Preservation, Rehabilitation Actions and Activities

	Flexible	Rigid	Surface Treatment
Routine Maintenance	• Pothole • Roadside maintenance • Drainage maintenance • Spray patching • Localized distortion repair	• Potholes • Spall repairs • Blow ups • Localized distortion repair	• Drainage improvement
Minor Rehabilitation / Preservation	• Rout & crack sealing* • Hot mix patching* • Surface sealing* (seal coat, slurry seal, micro-surfacing, chip seal/ surface treatment) • Texturization* (micro-milling, shot blasting, sand blasting) • Asphalt strip repairs* / full depth patching • Hot mix resurfacing* • Partial depth removal (milling) & resurfacing* • In-place recycling* (HIR, CIR, CIREAM) • Cold mix with sealing course* • Distortion correction • Drainage improvements • Frost treatments • Roadside slopes and erosion control	• Resealing and sealing of joints and cracks* • Load transfer retrofit* • Full depth joint repair* • Full depth stress relief joints* • Milling of stepped joints and distortion* • Subsealing & joint stabilization* • Surface texturization / diamond grinding*	• Spray patching* • Chip sealing* • Levelling • Full depth patching
Major Rehabilitation	• Full depth removal & resurfacing • Full depth reclamation / pulverization • Pulverization with expanded asphalt stabilization • Unbonded concrete overlays • Whitetopping	• Bonded concrete overlays • Unbonded concrete overlays • Cracking and seating (with resurfacing) • Rubblization with resurfacing • Widening and shoulder retrofits • Full depth slab repair* • Precast concrete slab repair* • Hot mix asphalt resurfacing* • Remove & replace concrete pavement	• Surface treatment (single and double) • Pulverization and resurfacing

Note: Strategies with an (*) asterisk are considered as pavement preservation strategies

4.2 FLEXIBLE PAVEMENT MAINTENANCE, PRESERVATION AND REHABILITATION

4.2.1 ROUTINE MAINTENANCE

Routine maintenance activities are carried out to address safety and operational concerns. Typical actions included in this category are pothole filling, roadside maintenance such as shoulder grading, grass cutting and snow removal, drainage maintenance, localized distortion repair and spray patching. These activities address the safety of the road for the driving public at all times and preserve the condition of the roadway.

Potholes

Potholes can be caused by a number of factors which includes [5]:

- Fatigue failure of pavements
- Durability issues related to joints
- Construction deficiencies such as segregation

These factors allow water to enter the pavement structure. Freezing temperature and traffic accelerate the formation of potholes.

Typical pothole repair techniques range from a basic “throw and go” method where either a hot or cold asphalt bound material is tossed by hand into the pothole and compacted using the supply truck wheels; to a more extensive treatment involving infrared heating of the affected area, saw cutting, tack coating and replacing with hot mix asphalt (HMA). Pothole patching can also be applied using a spray injection machine where the nozzle is first aimed at the pothole to blow out water and debris; then a tack coat is sprayed on the bottom and sides, followed by asphalt and high-quality aggregate which are mixed as they travel down the hose. Extensive research on various pothole repair material and placement techniques was completed as part of the Strategic Highway Research Program [6]. There are a number of proprietary patching products available. Provided the manufacturer’s recommendations are followed closely, many of these can be effective. Generally, these products are more expensive than normal hot or cold mix patching materials. Pothole repair is usually considered a short term maintenance treatment for safety purposes.

Detailed procedures for pothole filling are given in SP-001 and the Maintenance Quality Standard MQS-101 (for Concrete Pavement Surfaces) and MQS-102 (for Asphalt Pavement Surface) and MQS-103 (for Surface Treated Surfaces), including distress classification (severity), recommended maintenance treatment alternatives and procedures.

Roadside Maintenance

Roadside maintenance involves a number of activities, all of which contribute to user safety and preservation of the roadway. The shoulder provides lateral support of the pavement edge and accommodates pavement runoff. Hence, activities such as shoulder grading to remove any granular "berms" and eliminate severe lane/shoulder drop-offs, removal of snow accumulated during winter months from ploughing of the roadway, and grass cutting are necessary.

During summer months, encroaching vegetation along the road shoulders can act as an impediment to surface and subsurface drainage, as well as obstructing the sight lines of the road user. Snow accumulation on the road shoulder from ploughing should be regularly removed. If left in place differential frost penetration can occur in frost-susceptible soils with associated differential heaving of the surfaced portion of the road. The melting snow can act as a source of water to the pavement base and subgrade and contribute to weakening of the pavement structure.

Shoulder grading warrants and procedures are covered in the Maintenance Quality Standards MQS-201 (Gravel Shoulders) and MQS-202 (Hard Surfaced Shoulders) and SP-001. Maintenance Best Practice MBP-201 (Gravel Shoulders) and MBP-202 (Hard Surface Shoulders) are generally maintained in conjunction with maintenance of the travelled portion of the roadway.

Drainage Maintenance

Noting that water is the key contributing factor to premature pavement deterioration, routine maintenance of the roadside drainage facilities must given a high priority as a maintenance activity. Surface and subsurface drainage must be provided with a positive outlet via well defined, free flowing ditches and/or subdrains. Ditches should be regularly cleared of any debris and vegetative overgrowth that could block the flow of water. Drainage conduits such as culverts, and the outlets from pavement subdrainage systems should be regularly checked and cleared of any blockages. Drainage maintenance procedures are outlined in Maintenance Quality Standard MQS-501 to MQS-505. Refer to Chapter 2 regarding drainage concerns.

Spray Patching

Surficial distress such as segregation and raveling of HMA pavements and streaking of surface treatment can be routinely treated as a maintenance activity to prevent or reduce further advancement and propagation of these distresses.

Spray patching assists in binding and sealing the surface of the pavement. A typical spray patching operation involves cleaning of the patch area to remove any loose material, spraying a thin layer of asphalt emulsion and spreading an appropriate aggregate in the affected area either by hand or by a mechanized process.

Procedures for this activity, as well as suitable materials and manpower / equipment requirements are covered in the Maintenance Performance Standard M- 1004 [2]. In addition, MTO has completed evaluations of several methods of spray patching including evaluation of a mechanized spray patching machine [7]. This procedure is outlined in Special Provision No. 399F18.

Localized Distortion Repair

Typical examples of distortion include frost heaves, shoving / rippling and sagging/dishing, depressions/settlement. Localized distortion repair is used to strengthen locally weak sections of pavement or to address surface defects such as depressions, bumps, etc. The treatment of these localized distresses consists of either cold mix or hot mix patching completed at the local level. In this regard, the repair guidelines provided in the Maintenance Manual: Maintenance Quality Standards (MQS) should be followed for all localized distortion repairs.

4.2.2 MINOR REHABILITATION / PRESERVATION

Factors Affecting Selection of Rehabilitation Activities

Rehabilitation activities are critical to preservation of the roadway and in some cases may increased service life. Routine maintenance activities can be conducted immediately in response to localized problems identified in the field, or as preventive measures to reduce the occurrence of future and more severe problems. Rehabilitation is relatively high cost and requires a complete evaluation/investigation of the cause(s) of the pavement distresses.

As a first step in the evaluation, a complete visual assessment of the pavement condition must be completed as detailed in Chapter 5 and in accordance with Reference [8a]. A comprehensive evaluation/investigation may include geotechnical investigative work, skid and/or roughness testing, pavement profile determinations or deflection testing.

Rout & Seal Cracks*

Moisture infiltration into a pavement structure may result in freeze-thaw damage to the pavement surface materials or weakening of the subgrade. In turn, this reduces the structural capacity. Crack sealing is intended to provide an effective barrier to reduce or eliminate moisture infiltration from the pavement surface. The effectiveness of crack sealing is dependent on the selection of an appropriate sealant material, the preparation of the crack and the installation method chosen.

Routing and sealing of cracks consists of mechanically cutting a sealant reservoir, cleaning and drying with hot compressed air, and filling the groove with hot-poured rubberized asphalt (carried out in accordance with OPSD 508.01 to 508.02) followed by dusting with mineral dust to prevent tracking when required. This activity is best carried out as the cracking becomes apparent and within 2 to 5 years of the pavement being placed. A given section of roadway may require successive treatments within this period to ensure that all significant cracks are properly sealed.

The recommended standard rout geometry is 40 mm wide by 10 mm deep geometry, but 20 mm by 20 mm geometry has been used in some applications. The use of a 20 mm rout geometry is discouraged as the operator of the router has more difficulty following a meandering crack. Furthermore, the 10 mm depth geometry allows the sealant to stretch more like a rubber band than a 20 mm deep one does. Currently, the MTO is looking at the possibility of using rout dimension of 30 mm wide by 15 mm deep and other alternative rout cross sections. A 10 mm by 10 mm rout dimension may also be employed where controlled cracking is necessary such as above joints or cracks in underlying concrete base.

Through evaluations starting in 1980, the MTO has considered a variety of sealant products for use in either or both HMA and concrete pavements (as covered under Designated Sources for Materials No. 3.05.40), routing equipment [9] and rout geometric configurations [10]. A detailed description of the MTO experience with routing and sealing of cracks in HMA pavements may be found in Reference [11]. The 40 by 10 mm rout is to be used for cracks up to 15 mm in width. For 15 to 30 mm width cracks, the crack is cleaned, dried and filled with sealant. Beyond that partial or full-depth crack repairs is recommended. All cracks are supposed to have an overband (or at least overfilled).

A given section of roadway may require successive treatments to ensure that all significant cracks are properly sealed. A video has been prepared by MTO describing in detail the proper method of crack sealing. As well, the economic benefits of timely crack sealing have been illustrated in a subsequent MTO study [12]. Crack sealing is also addressed in the Strategic Highway Research Program's (SHRP) Maintenance Manual for asphalt pavements [6].

Hot Mix Patching*

Hot mix patching consists of manual or machine placement of hot mix asphalt (HMA) to strengthen weak sections or to smooth out major surface defects such as depressions, bumps and other distortions. In hot mix patching operations, particular attention should be given to preparation of the existing surface noting that it must be clean and dry. Depending on the traffic volume, consideration should be given to cold planing/milling the existing HMA surface at the end treatments in order to have a smooth transition between the hot mix patch area and the existing roadway. This will also promote improved bonding at the patch ends. Regardless, tack coat should be properly applied at the patch edges and end treatments.

Hot mix patching is to be completed in accordance with OPSS.PROV 313 "Construction Specification for Hot Mix Asphalt – End Result" and OPSD 517.01.

Surface Sealing*

Surface seals required to maintain performance characteristics of the wearing surface generally consist of chip seals, seal coats, slurry seal, micro-surfacing and fog seals.

A chip seal treatment consists of spray application of emulsified asphalt (RS-2, CRS-1, CRS-2 or HF emulsions specified in OPSS 1103) followed by spreading of uniformly graded crushed stone (single sized) aggregate in accordance with Maintenance Performance Standard M-1017 [2]. Proper chip sealing produces an all weather surface that helps seal the pavement and provides improved skid resistance.

In addition to the application procedure described in Section 4.2.1 on Spray Patching, there are two methods of applying surface sealants:

1. An automated surface sealing system such as Dynapatch, comprises of a controlled

spray applicator (variable width), aggregate spreader (also variable width) and compaction system, all contained within a single travelling machine.
2. Conventionally applied emulsified asphalt (spray truck) followed by an aggregate spreader and conventional asphalt pavement compaction equipment.

A seal coat treatment is a chip seal application that can be placed in single or double layers. Seal coat is the process of placing emulsified asphalt coating on the existing road surface. A layer of aggregate is then applied and rolled into the emulsion (Figure 4.2.1). Additional aggregate and emulsion is placed in a double layer. Seal coat has been used by MTO as a pavement preservation treatment to seal the existing pavement surface to improve ride and skid resistance. Another similar process with fiber modified chip seal consists of a traditional double seal coat with fiber-reinforcement in the bottom emulsion layer. The fiber-reinforced membrane is designed to delay reflective cracks and seal alligator cracks. A trial section was built in Eastern Region to evaluate this new surface sealing technology.

Figure 4.2.1: Double Seal Coat Application

A slurry seal consists of an asphalt emulsion mixed with fine aggregate and is usually applied in a very thin application over the full width of the pavement. It is used to assist in preventing moisture ingress through the pavement surface and to improve skid resistance properties. Slurry seal is used more commonly for low volume roads. See OPSS 337 for details.

Micro-surfacing is a cost effective maintenance technique, provided that pavement deterioration is still well above the minimum acceptable level. Micro-surfacing mixture generally consists of a polymer modified asphalt emulsion, medium to fine graded high quality aggregate, filler, additives, and water. It is placed by specialized equipment in lifts of 8 to 10 mm. Micro-surfacing can be used to address pavement rutting or other surface deficiencies by providing a new riding surface and increased friction. See OPSS 336 for details.

Fog seals consist of diluted emulsified asphalt (maximum dilution rate of 20% is recommended) applied to asphaltic concrete surfaces that are segregated, oxidized or low in asphalt cement. A light second application may be provided if necessary. Thicker applications reduce frictional properties.

Fog seals and slurry seals are rarely used within MTO but could be considered where surficial ravelling or segregation requires that only moderate void filling is necessary to mitigate further problems and improve the pavement texture. Detailed discussions on these surface sealing techniques are given in the MTO Manual for Condition Rating of Surface-Treated Pavements –Distress Manifestations, SP-021.

Texturization*

Surface texturization is sometimes used to improve the frictional properties of pavement surfaces. The most common method of texturizing an HMA surface is to abrade it with high velocity steel shot (shot blasting). Sand blasting is a similar process using other types of abrasive media (aluminum oxide, silicon carbide, glass beads etc.). A new technology called water blasting is being evaluated as a technique to remove excess bitumen from the pavement surface.

Micro-milling is also a texturization method to improve frictional properties of HMA surface. It uses a milling machine with teeth but spaced more closely and the milling depth is shallower.

Asphalt Strip Repairs*

Asphalt strip repairs can be undertaken to treat moderate to severe longitudinal cracking, centre line cracking, transverse cracking, random cracking, and opening along the interface between concrete pavements and HMA shoulders. The procedure generally involves partial grinding or milling of the HMA surface to a width of about 0.3 to 1.0 m and a depth of at least 50 mm, and subsequent replacement with HMA compatible with the existing pavement surface course. Specific maintenance standards have not been developed for this procedure, but it has been successfully completed regionally within the MTO.

Hot Mix Resurfacing*

Hot mix resurfacing is the principal rehabilitation alternative being employed on Ontario highways. All hot mix resurfacing work is to be completed in accordance with OPSS.PROV 313 and OPSS.PROV 1151. The overlay thickness required will depend on the ride and structural condition of the existing roadway as discussed in Section 3.3.3. Prior to resurfacing, padding or milling may be necessary when settlements and distortions of the road surface preclude placement of a smooth overlay.

The ride and/or structural condition of the roadway may require a substantial overlay thickness to satisfy minimum pavement thickness and deflection criteria. In this case, it may be more practical to place granular padding directly over the existing surface in order to reduced thickness of hot mix resurfacing. The decision to place granular padding will depend largely on the thicknesses involved and the traffic loads. But it is important to note that traffic shear forces can cause slippage to occur along the interface of granular padding and the HMA surfaces. In high traffic situations, the use of bituminous treated base may be considered as an alternative.

Partial-Depth Removal and Hot Mix Resurfacing*

The structural condition of the pavement may be acceptable; however aging of the surface or surficial defects such as ravelling, segregation and stripping may dictate partial-depth removal and replacement of the asphaltic concrete with HMA surfacing. Partial-depth removal and replacement may be necessary where the existing curb and gutter, catchbasins or drainage will not permit raising of the pavement surface.

Partial-depth removal, also called milling, is normally accomplished by using cold milling equipment. The material removed from the existing pavement surface may be transported to a central plant for recycling in accordance with OPSS.PROV 1151. The milled material may be used for recycled HMA mixes blended with natural aggregate or reclaimed crushed concrete and used in granular base and subbase or use for shouldering. Milling and resurfacing should only be used if the underlying HMA layers are sound.

In-Place Recycling*

In-place recycling involves scarification of the existing pavement surface, in-place processing of the scarified materials, possibly adding binder (with or without additives to improve the binder properties) and simultaneous placement of the processed material. This operation can be completed by several different methods and various types of equipment, either hot or cold. Regardless of the process selected, candidate sections for in-place recycling should be thoroughly evaluated. The properties of the existing HMA dictates the properties of the finished product (aggregate characteristics and asphalt cement properties in particular). For example, if the existing surface exhibits distresses associated with aggregates performance, an in-place recycled pavement will also exhibit similar problems.

Cold in-place recycling (CIR) is an established pavement rehabilitation method that mills an existing HMA pavement, processes the millings by screening and secondary crushing, mixes in additional asphalt cement, and places the final mixture in a windrow ready for spreading and compacting without off-site hauling and processing. The additional asphalt cement is typically an emulsion. CIR material requires application of a wearing surface to seal the material. The Asphalt Institute's Manual "Asphalt Cold-Mix Recycling" [13], summarizes the general procedures involved and discusses the problems that can occur. Also, Reference [14] illustrated Ontario's experience with CIR. Emulsified asphalt cold mix pavements are generally addressed by Ministry Directive C-55.

Figure 4.2.2 Typical Cold In-Place Recycling Train Unit

A further development in CIR technology is the use of expanded (foamed) asphalt cement, rather than emulsified asphalt to bind the mix. This combination of CIR and expanded asphalt technologies is termed Cold In-Place Recycled Expanded Asphalt Mix (CIREAM). With conventional CIR, ministry specifies a minimum 14-day curing period to allow the emulsion to set. Moisture and compaction requirements also need to be met prior to placing a new wearing surface course. Application of CIR is usually limited to the warmer, drier months [15]. The advantage of CIREAM is that a new HMA surface can be applied after 3 days, and the process is less dependent on warm, dry weather for placement.

Figure 4.2.3 Typical Cold In-Place Recycling Expanded Asphalt Train Unit

Hot in-place recycling (HIR) involves heating and scarification of the pavement surface, in-place processing of the scarified materials including the possible addition of beneficiating materials such as rejuvenators (ie, emulsion), asphalt cement, aggregates or new HMA followed by screeding and compaction of the reprocessed materials in one continuous operation [16]. Benefits of this type of operation include the reuse of the existing pavement materials, treatment of existing pavement surface distresses and the ability to reprofile as well as rejuvenate the pavement surface. Integral overlays (ie, two screeds on one machine) can be placed and compacted simultaneously. This method is only suitable if the pavement surface is free of major structural distress.

Figure 4.2.4 Typical Hot In-Place Recycling Train Unit

Figure 4.2.5 Hot In-Place Recycling Train Coil Pre-heater Unit

Figure 4.2.6 Hot In-Place Recycling Train - Martec AR-2000 on Highway 401, Ontario [17]

Cold Mix with Sealed Surface Course* (Midland Paver)

Cold laid mixes are produced either through a Midland Mix Paver or a central plant. The Midland Mix Paver comprises a mobile plant which mixes and places aggregate and emulsified asphalt. Aggregate is delivered to the site by truck and dumped into the paver hopper, similar to a conventional hot mix paver. The aggregate is conveyed to a pugmill where it is sprayed with emulsified asphalt at a metered rate. The mix is then distributed across the roadway via rear augers and levelled using a vibratory screed. The open-graded mix tends to ravel under higher traffic volumes and requires a wearing course over the cold mix surface to seal the cold mix similar to CIR material. The construction of pavements using the Midland Mix-Paver process is discussed in detail in Construction and Performance of Cold Mix Placed By the Midland Mix-Paver [18] and Ministry Directive C-55.

Distortion Corrections (Grinding, Milling, Heating and Raking)

Distortion of the HMA pavement surface can be handled in several ways in addition to conventional hot mix patching as previously discussed. Removal of distorted pavement by cold planing and heater planing are discussed in SP-001 [2]. Recent equipment advances involve heating of the pavement surface using infrared heating panels that do not damage the asphalt to the same degree as heating with direct flame. In either case, the milled or scarified surface may be covered with an HMA patch or surface treated to provide a smooth riding surface.

Drainage Improvements

Good drainage is the single most important feature to ensure the pavement can withstand the effects of weather and traffic. Drainage improvements include items such as major reditching, installation of curb and gutter, subdrain installation or retrofits. The roadside drainage must be well maintained in order for the surface runoff to a positive outlet. Provision of subdrains and/or curb and gutter construction is discussed in Chapter 2.

Frost Treatments

The treatment of frost heaves, and major road improvements necessary to mitigate pavement damage due to frost, are well documented within MTO and are discussed in detail in Section 3.2.5. OPSD-514-010 and OPSD-514-020 provide design guidance for the use of expanded extruded polystyrene insulation to limit frost penetration, and OPSD-205-06 describes the procedures to be followed for the excavation of frost heaves. All frost heaves and boils should be investigated by the Regional Geotechnical Section.

Roadside Slopes and Erosion Control

The Regional Geotechnical Section would be consulted to investigate where continuous problems with sloughing of embankments and/or damage (cracking or subsidence) of the roadway adjacent to sloped areas are observed. The maintenance actions will be carried out in accordance with good geotechnical engineering practice (slope stabilization by installation of retaining structures, reducing side slopes, etc.) as covered in section 2.4 when the overall stability of the slope or embankment is proven to be a problem. However, localized problems such as wash outs and shallow slippages due to improper vegetation may be easily treated through excavation and replacement with geotextile and granular material.

Maintenance Quality Standards MQS-323 provides guidance with regard to vegetation maintenance and regeneration. Erosion problems associated with runoff may be treated by altering the general drainage characteristics of the roadway, or by localized provision of erosion resistant materials or treatments. For example, rip rap and geotextile-lined spillways may be placed where infrequent but locally severe runoff causes damage to the side slopes.

4.2.3 MAJOR REHABILTATION

Major rehabilitation of the pavement will be necessary when the pavement serviceability or PCI (Pavement Condition Rating) is lower than the minimum acceptable level for a given road classification, and routine maintenance or minor rehabilitation activities are not able to adequately address the problems. The recommended rehabilitation alternatives for each road will be dependent upon the riding quality and the type of distresses, causes and severity. Financial, operational and planning are also considered. Regardless, a thorough investigation of the pavement condition is required to determine the nature and extent of the pavement problems in order to decide on the optimal rehabilitation alternative.

Full-Depth Removal and Resurfacing

Full-depth removal and replacement with HMA should be considered when the existing HMA is too distressed or too thin (approximately 80 mm) to be considered for in-place recycling. The existing asphalt is removed by a variety of methods including milling, ripping, etc. and transported to a central asphalt plant site. The existing HMA is crushed at the plant site to the appropriate size, then stockpiled for recycling in the asphalt plant. Ministry Directives C-129, C-145 and C-165 and Section 1.4.1 all pertain to the use and production of recycled HMA pavements.

The exposed granular base can be reshaped after removing the asphaltic concrete, strengthened with additional granular base as necessary, and then placed with HMA.

Full Depth Reclamation (Pulverization) with Resurfacing

This rehabilitation technique is often used for pavement exhibiting extensive distress. It involves pulverization of the pavement surface layers and a portion of the underlying granular base for a total depth of up to 300 mm. The desired reclaimed asphaltic concrete and granular (coated to uncoated particle) blend is a 50:50 (or higher percentage for granular), in other words, the pulverized mix should contain a maximum of 50% coated particle. The resulting mixture used becomes the new granular base or subbase and is overlain with a new riding surface. The pulverized mix can also be stabilized using bituminous materials, hydraulic cement, lime or calcium chloride. Refer to Section 1.3 of this Manual for more details. A new granular base material can be added to improve the structural capacity of the pavement and is overlain with a new riding surface. Advantages of this technique include the reuse of the existing pavement materials and the elimination of potential reflection cracking from an old HMA layer through the new surface.

Full Depth Reclamation with Expanded Asphalt Stabilization (EAS)

A further development in Full Depth Reclamation is Expanded Asphalt Stabilization (EAS), which uses the same pulverization process except that expanded (foamed) asphalt is added to stabilize the material (Figure 4.2.7). In the process, a small amount of water is added to the hot asphalt cement, which vaporizes causing it to rapidly “foam” and uniformly coat the aggregate particles with asphalt cement. Refer to Section 1.3.2 of this Manual for more details.

Figure 4.2.7 Typical Expanded Asphalt Stabilization Process

Unbonded Concrete Overlays

In this method of resurfacing, the new HCC (PCC) slab is placed over an existing slab or asphaltic concrete pavement such that it can move independently of the underlying structure. To accomplish this, a bond breaking material is placed between the two lifts to ensure that bonding does not occur. Materials typically used as bond breakers are two applications of a sand seal, double applications of wax based curing compounds, a thin lift of SuperPave 4.75 (sand mix), plastic sheeting, roofing felt or a slurry seal. Typically unbonded concrete overlays are 150+ mm in thickness with joints offset from those in the existing pavement, thereby improving load transfer efficiency.

Whitetopping

This is a technique involving hydraulic cement concrete where existing HMA pavement is overlaid with a structural layer of concrete. This thin-concrete overlay - whitetopping is a structural layer that is placed on the milled asphalt which is typically 50 to 100 mm thick. It is typically used to address rutting problems with the existing hot mix pavement. More information on this technique may be obtained from the Canadian Portland Cement Association (**http://www.cement.ca/**) and the American Concrete Pavement Association (**http://www.pavement.com/**).

4.3 RIGID PAVEMENT MAINTENANCE, PRESERVATION AND REHABILITATION

4.3.1 ROUTINE MAINTENANCE

Routine maintenance of rigid pavements involves activities such as spall repairs, blow-ups and distortion repairs together with modifications for the specific materials involved. Rigid pavements consisting of exposed hydraulic (Portland) cement concrete (HCC or PCC) and composite pavements (HMA over concrete). For composite pavement, it requires some of the same maintenance activities as flexible pavements. Roadside maintenance activities for rigid pavement are virtually identical to flexible pavements.

Potholes (Composite Pavements)

Pothole repairs for composite pavements are to be completed in accordance with SP-005 [19] and Maintenance Quality Standard MQS-101 to MQS-103. In addition, SP-001 [2] also provides guidance for pothole filling for rigid pavements.

Spall Repairs

Spalling consists of the breaking or chipping of the pavement at joints or cracks, usually resulting in fragments with feathered edges. Spalling is progressive and it is generally due to excessive local pressure caused by a combination of traffic, debris trapped in joints and cracks, thermal expansion and thermal gradient curling (warping) of the concrete slabs. The treatment for spalling consists of saw cutting the perimeter of the removal area and removing all the spalled concrete with a chipping hammer. For temporary repairs, the removal area will be completed using cold mix. For long term spall repair, a full road closure is required to fill the removal area with concrete; and for fast track repair, the removal area can be filled with proprietary patching material.

Blow-ups

Blow-ups are pavement failure due to high horizontal compressive stresses resulting from thermal expansion of the slabs, restrained joints and/or locked dowels. Failure is sudden and very severe requiring complete removal of the failed concrete and replacement with cast-in-place concrete or temporary hot mix repairs as covered by Maintenance Performance Standard M-1012 [2], Maintenance Best Practice, MBP-101 [1]. In the event that the contractor performing the maintenance work considered joint failure to be imminent, full-depth stress relief joints should be installed as discussed in Section 4.3.2.

Localized Distortion Repair

Distortion corrections can be handled in several ways. Composite pavements should be repaired in a similar fashion as flexible pavements as discussed in Section 4.2.1. Distortion repairs for exposed concrete pavements should be completed by cold planing (Maintenance Performance Standard M-1090C [2], Maintenance Best Practice, MBP-101 [1]) and machine placed HMA (Maintenance Best Practice, MBP-102 [1]).

4.3.2 MINOR REHABILITATION / PRESERVATION

Resealing Joints and Sealing Cracks*

Joint sealant loss can occur with time as a result of traffic action, improper sealant installation, deterioration of the sealant material or expulsion by compressive forces closing the joint. After removal of the existing joint sealant and cleaning of the joint with a hot air lance, joints should be resealed using hot poured rubberized joint sealing compound in accordance with OPSS 1212 and DSM No. 3.20.45. Where joints are open to their full depth, a backer rod shall be placed into the joint prior to resealing to control sealant penetration depth. Refer to reference [20] for more details.

Cracks in exposed concrete or composite pavements may be treated as previously discussed in Section 4.2.2. It should be noted that the Designated Sources List has separate listings for sealants approved for use in asphaltic concrete and hydraulic cement concrete (HCC) pavement (DSM No. 3.05.40 for asphaltic pavement and DSM No. 3.20.45 for concrete pavements).

Cross-stitching longitudinal cracks is a method to repair low-severity cracks. This method adds reinforcing steel to hold the crack together tightly in a diagonal stitching pattern to stop further movement of the concrete pavement to prevent opening of the cracks.

Load Transfer Retrofit*

Pavement exhibiting loss of load transfer, usually identified by joint faulting or FWD testing, can be stabilized by having load transfer restored or augmented. Pavements requiring improved load transfer can be retrofitted with dowels or other devices that are placed in milled out slots cut parallel to the direction of traffic. It is done through cutting slots into the pavement. Where voids exist underneath the pavement, the slab may also require stabilization. Successful restoration requires sound concrete, and badly cracked or spalled concrete should be considered for full depth repair (ie, complete replacement of the joint). After load transfer restoration, the entire surface is usually ground to restore ride quality [21].

Full-Depth Joint Repair*

Joint failure can occur as a result of either high deflections of the slab edges due to lack of sufficient support, or pavement blow-ups (previously described in Section 4.3.1). The remedial action may consist of excavation and placement of cast-in-place concrete (Maintenance Performance Standard M- 1012 [2]), or granular placement with HMA patching (as covered by Maintenance Performance Standard M-1001A* [2]).

Full-Depth Stress Relief Joints*

Full-depth stress relief joints can be installed to prevent serious joint failures (blow-ups). The remedial action consists of cutting the concrete pavement slab to its full lane width and depth for a longitudinal length of two metres (minimum), removal of concrete, placement of granular base to within 150 mm of the pavement surface, and placement of compacted HMA (2 lifts 75 mm thick). An alternative used in the past, referred to as Vermeer Stress Relief Joints is covered by Maintenance Performance Standard M- 1090V [2].

Milling of Stepped Joints and Distortions*

Stepping (faulting) is the vertical displacement of abutting slabs at joints and cracks, creating a 'step' deformation in the pavement surface. The 'approach' slab is typically elevated in relation to the 'leave' slab. Stepping is caused by pumping of moisture and fines beneath the slabs resulting in the formation of voids and loss of slab support. Removal of stepping is accomplished by either transversely cold milling a 0.5 m wide strip along the crack or joint where the difference between the two slabs after milling is not greater than 3 mm or; milling the lane full width against the direction of traffic. The equipment used must have accurate grade and slope control, as well as a means to control dust. Upon completion of the milling operation, the pavement must be swept to remove any loose materials. This treatment only removes the step and does not eliminate the pumping and possible future stepping. A comprehensive subsealing program is required to stabilize the joint prior to milling.

Subsealing and Joint Stabilization*

Slab rocking or pumping can result in formation of voids at the joints or cracks in concrete pavements. Subsealing (joint stabilization) of the pavement should be considered in order to stabilize the slabs prior to resurfacing or to prevent further slab cracking due to flexure.

A cementitious grout mixture (3 parts flyash to 1 part hydraulic cement) or urethane is injected into the voids through a predetermined hole pattern drilled adjacent to the joints and cracks. The grout pressure and slab elevation during grouting are carefully monitored to ensure that the voids are just filled, and the slab is not lifted to any significant degree. Subsealing has been successfully completed on a contract basis during the rehabilitation of Highway 401 in Toronto [22]. A recommended specification for subsealing is given for this project in Reference [22].

Surface Texturization* (Diamond Grinding)

Surface texturization can be considered as a rehabilitation technique to improve pavement skid resistance and ride quality of a concrete pavement surface, as well as to promote better surface drainage. This can be achieved through diamond grinding or by abrading the surface of the pavement with high velocity steel shot [23].

In addition, diamond grinding can be applied to reduce pavement noise. Diamond grinding equipment consists of a grinding head approximately 1 m in width and having about 60 diamond grinding blades per 300 mm. The grinding equipment is self-propelled and specifically designed to texture concrete pavements. Diamond grinding is generally completed in the longitudinal direction to remove slab stepping and localized distortions, improve skid resistance, and reduce noise. Typically less than 5 mm of concrete is removed from the pavement surface by the diamond grinding equipment. Care must be taken to ensure that the removal of concrete does not significantly affect the structural capacity of the pavement.

Transverse grooving of an exposed concrete pavement is conducted to promote surface drainage, and facilitate the removal of water that could contribute to vehicle hydroplaning. Specifically designed machines equipped with circular diamond grinding blades groove the pavement at a random spacing of 25 mm maximum and to a depth of about 10 mm. Although several contracts involving grooving have been completed, this technique is no longer used due to the excessive tire/pavement noise produced.

HMA Resurfacing*

HMA resurfacing is used to improve the ride quality and frictional properties, and reduce maintenance costs of the existing pavement. Repairs are first made to higher severity distresses and the pavement is tack coated and resurfaced with one or more layers of HMA. Typical HMA overlays are 60 to 100 mm in thickness.

4.3.3 MAJOR REHABILITATION

Detailed rehabilitation design and construction guidelines, as well as specifications for concrete pavements are given in Reference [21]. It is considered to be an excellent reference. Rehabilitation treatments on concrete pavement are usually designed to extend the service life of the pavement by 12 years or more.

Bonded and Unbonded Concrete Overlays

For bonded concrete overlay design, the concrete overlay is bonded positively to the existing cleaned concrete. It is done by shot blasting the existing surface followed by applying a grout along the pavement interface. By bonding the new slab (up to 75 mm thickness) to the old slab, the resulting thicker section is structurally more able to carry heavy loads and the pavement service life is extended. Bonded concrete overlays are generally used in those situations where the existing concrete slab is in relatively good condition but it is desirable to improve the profile and load carrying capacity of the total pavement section. It is necessary to match the new and existing joints or reflection cracking in the new overlay. The 1981 PCA publication Guide to Concrete Resurfacing Designs and Selection Criteria is given as a specific reference in this regard [24].

For unbonded concrete overlay, refer to section 4.2.3.

Cracking and Seating (with Resurfacing)

Cracking and seating of severely cracked and distorted concrete pavements involves destructively breaking the slab using specifically designed breaking equipment (modified pile driver, Wirtgen guillotine or vibratory technique, etc.). Then seating the broken slab with a heavy roller to eliminate slab rocking and restore full contact between the slabs and underlying substrate, i.e., eliminate voids. The existing concrete slab can either be completely broken up or selectively cracked to a maximum 600 mm crack pattern to restore full contact. Once cracked and seated, the pavement can be resurfaced using either HMA or concrete pavement. Results to date suggest that this technique only marginally prevents reflection cracking through the overlay. An improved process called rubblization is used to replace crack and seat, see next section for details.

Rubblizing and Resurfacing

Rubblizing is a similar process to cracking and seating, where the old concrete pavement is further broken into pieces 100 mm or smaller, using specialized equipment such as a resonant breaker. This process is very useful in recycling reinforced pavements, as the reinforcement can be separated from the concrete during the rubblization process, making further processing much easier. The old concrete can be left in place and compacted as a base and overlaid with either HMA or concrete pavement.

Widening and Shoulder Retrofits

Widening of existing concrete pavements and concrete shoulders should match the existing concrete pavement thickness. The corresponding granular base and subbase thicknesses should also be considered. The widening or shoulder must be positively connected to the existing concrete pavement using tiebars installed at regular intervals. In order to limit stresses in the concrete pavements, the joint spacing and dowelling in the widening or shoulder section should match that of the adjacent lane.

Full-Depth Slab Repair

Full-depth slab repair consists of removal and disposal of the existing damaged concrete pavement, base restoration, supply and installation of tie bars, dowels and/or load transfer devices, and supply and placement of concrete. The Portland Cement Association Information leaflet Patching Concrete Pavements [25] is an excellent reference in this regard.

Concrete supply and placement shall be in accordance with OPSS 350, 360, 362 and 364, with the materials as required by OPSS.PROV 1350. Pavement sections should satisfy OPSD 216.02 for composite pavements on granular base, or OPSD 207.01, 207.02 or 207.03 for concrete or composite pavements over lean concrete base or cement treated base. In situations where the roadway must be opened to traffic as soon as possible, fast-track (or rapid setting) concrete may be employed. Calcium chloride has been successfully used in combination with Type 30 (High Early) hydraulic cement to achieve compressive strengths in excess of 20 MPa within 5 hours [26].

Precast Concrete Slab Repair

Precast concrete slab repair is part of full depth concrete slab repair but specifically using pre-cast slab panels to speed up the construction process. It is especially useful for the rapid repair and replacement of concrete pavement. Precast concrete slabs (including pre- and post-tensioned slab systems) are pre-engineered concrete pavements that are fabricated off-site and transported to the project site for installation on a prepared foundation. These systems do not require field curing time to achieve strength and durability. They can often carry traffic immediately after installation. This minimizes closure times and accompanying traffic delays that result from detouring or staging. In addition, precast slabs can be manufactured with strict control of materials and environmental conditions, which maximizes the potential for long-term durability and pavement life.

In November 2004, MTO conducted a trial project to evaluate construction techniques for pre-cast concrete pavement slab repairs. The trial, carried out on the heavily trafficked Highway 427, constitutes the first use of this repair technology in Canada [27]. More precast concrete slab repairs were done along Highway 427 after the first trial in 2004. The pre-cast repairs are similar in both ride and appearance to fast-track repairs along the same section of highway. MTO will continue to monitor the field performance of these innovative pre-cast technologies, and will assess the cost effectiveness of this alternative to fast-track concrete repairs.

Figure 4.3.1 Precast Concrete Slab Repair on Highway 427

4.4 SURFACE TREATMENT MAINTENANCE, PRESERVATION AND REHABILITATION

4.4.1 ROUTINE MAINTENANCE

Drainage Improvement

Drainage is very importance to the pavement structure of a surface treated road. Majority of these pavements are found in rural areas, drainage improvements usually consist of improving and deepening free flowing ditches, or retrofitting subdrains. All pavements will benefit from this improvement at almost any level of distress.

4.4.2 MINOR REHABILITATION / PRESERVATION

The common minor rehabilitation or perseveration treatments for surface treated roads involve spray patching, chip sealing, leveling and full depth patching. The effectiveness of deep patching depends on the structural capacity provided by the patch but is normally expected to have a service life of up to about 5 years.

Spray Patching*

Surface distresses such as ravelling or aggregate polishing can be treated by spray patching to prevent further deterioration of the pavement and to improve skid resistance. A typical spray patching operation involves cleaning of the patch area to remove any loose material, spraying a thin layer of emulsified asphalt and spreading of an appropriate fine aggregate in the affected area. Some agencies, such as New Brunswick, also spray patch shallow pot holes (or fill them with HMA depending on severity).

Chip Sealing*

In this rehabilitation procedure, a thin layer of emulsified asphalt is sprayed onto the pavement surface in areas of localized distress followed by spreading a single layer of single sized aggregate with a small mechanical spreader. The emulsion and aggregate is then compacted with a mechanical compactor. This treatment is used to limit localized surface ravelling and to prevent moisture from infiltrating the pavement through surface distresses.

Leveling*

Leveling usually consists of the placement of Granular A or cold mix asphalt on the existing surface treated pavement. Leveling is used to correct minor longitudinal profile deficiencies, cross-fall deficiencies, or to locally improve the structural capacity of the pavement. Some agencies, such as New Brunswick, also do a significant amount of levelling with HMA.

Full Depth Patching*

Full depth patching for surface treated roads is usually applied to pavement exhibiting medium to high severity load-related distresses such as rutting, distortion or alligator cracking. The surface treatment, granular base and any deleterious subgrade material such as muskeg are excavated from the repair area. New granular base material is placed and properly shaped and compacted, followed by the placement of a new surface treatment or HMA pavement surface.

4.4.3 MAJOR REHABILITATION

There are two methods for major rehabilitation of surface treated roads. These rehabilitation strategies are applying single or double surface treatment; and pulverizing followed by single or double surface treatments. These rehabilitation strategies have a service life of 5 years or more.

Surface Treatment (Single and Double)

A rehabilitation surface treatment involves the application of a thin layer of emulsion followed by a single layer of aggregate; refer to as a single surface treatment. The thickness of the treatment is about the same as the nominal maximum size aggregate particles. This is often followed by an additional application of emulsion and aggregate which is referred to as a double surface treatment. A surface treatment is used to provide a bound riding surface and to act as a barrier against moisture infiltration.

This type of treatment may also involve localized repairs followed by additional layers of reapplication, or it may involve a leveling first, followed by a reapplication.

Pulverization or Scarification and Resurfacing

This rehabilitation treatment is normally used for surface treated roads exhibiting extensive high severity distress. It involves pulverization or deep scarification of the pavement surface layers and a portion of the granular base for total depths up to 300 mm. The mixed old surface treatment materials and granular base can then be reshaped and compacted and used as a granular base or subbase followed by a double surface treatment. The surface and base can also be stabilized with bituminous material or hydraulic cement. New granular base material can also be added to improve the structural capacity of the pavement followed by the placement of a new riding surface. Advantages of this treatment include the reuse of the existing pavement materials and the elimination of potential reflection cracking from the old surface layer.

4.5 SUSTAINABLE PAVEMENT REHABILITATION STRATEGIES

The ministry is committed to help build a more sustainable transportation system that supports today's needs while protecting the environment for future generations. The ministry has implemented an innovative in-situ pavement recycling program to provide a sustainable rehabilitation option that is safe, efficient, environmentally friendly and cost-effective, that meets the needs of present-day users without compromising those of future generations.

A sustainable pavement can be defined as a safe, efficient, economic, environmentally friendly pavement meeting the needs of present-day users without compromising those of future generations. The main criteria established for a sustainable pavement are:

- Minimizing the use of natural resources
- Minimizing energy consumption
- Minimizing greenhouse gas emissions
- Limiting pollution
- Improving health, safety and risk prevention
- Ensuring a high level of user comfort and safety

With the increasing cost of fuel and environmental awareness, pavement recycling has become a preferred choice when selecting rehabilitation strategies for Ontario's highways and is frequently replacing traditional paving techniques. MTO has been recycling pavements since the late 1980s. The recycled pavements have been monitored and evaluated over the years in order to develop improved specifications and cost-effective processes. The following table provides a list of the pavement recycling processes used by the ministry and the quantities of recycled material that each has generated.

Table 4.5.1 Pavement Recycling Efforts in Ontario (until 2011)

Recycling Techniques	Year (first started)	Recycled Quantities
General (Central Plant) Recycling	1988	5,834,000 tonnes
Full Depth Reclamation (FDR)	1988	50,272,000 m^2
Cold In-place Recycling (CIR)	1989	7,065,000 m^2
Hot In-place Recycling (HIR)	1990	3,553,000 m^2
Expanded Asphalt Stabilization (EAS)	2000	2,604,000 m^2
CIR with Expanded Asphalt (CIREAM)	2003	3,635,000 m^2

To date, CIR and CIREAM are the most cost effective, socially conscious and environmental friendly pavement rehabilitation option. The ministry will continue to contribute to the protection and enhancement of the environment through the implementation and promotion of innovative, sustainable pavement recycling techniques [28].

4.5.1 MTO GREEN PAVEMENT RATING SYSTEM - GREENPAVE

Environmentally-friendly pavement design, preservation and rehabilitation strategies have a strong focus on recycling and re-use of road pavement materials (RAP, concrete, granular), industrial co-products (blast furnace slag, fly ash and silica fume) and diverted waste stream products (roof shingles, recycled glass and ceramics).

In order to encourage the use of sustainable pavement rehabilitation strategies, MTO developed an Ontario based Green Pavement Rating System - GreenPave to quantify and encourage pavement sustainability. There are a few existing green rating systems readily available or under development. GreenPave is primarily based on the Green Roads [29] (Washington State) and GreenLITES (New York State) rating systems [30], but customized for Ontario. The LEED® certification program and Alberta's rating system [31] were also assessed during the development. The main difference between MTO's GreenPave rating system and other rating systems is GreenPave focuses specifically on the pavement component rather than the entire road.

Using a simple, points based rating system, GreenPave is designed to assess the "greenness" of pavement designs or constructed pavements, both flexible and rigid structures. Assigning a rating to the pavement design will enable MTO to incorporate more sustainable technologies in pavements and encourage industry to do the same. Assigning a rating to constructed pavements eliminates design assumptions and allows for incorporation of construction components and contractor innovation that cannot be estimated at the time of design.

In the GreenPave rating system, pavements will be assessed within four sustainability categories (Table 4.5.2).

Table 4.5.2 Four Sustainability Category in GreenPave

Category	**Goal**	**Points**
Pavement Design Technologies	To optimize sustainable designs. These include long life pavements, permeable pavements, noise mitigating pavements, and pavements that minimize the heat island effect.	9
Materials & Resources	To optimize the use/reuse of recycled materials and to minimize material transportation distances.	11
Energy & Atmosphere	To minimize energy consumption and GHG emissions.	8
Innovation & Design Process	To recognize innovation and exemplary efforts made to foster sustainable pavement designs.	4
	Maximum Total:	**32**

Each category is further broken down to address specific objectives, with corresponding points assigned to each subcategory. Each category is divided into two to four subcategories as illustrated in Figure 4.5.1 below.

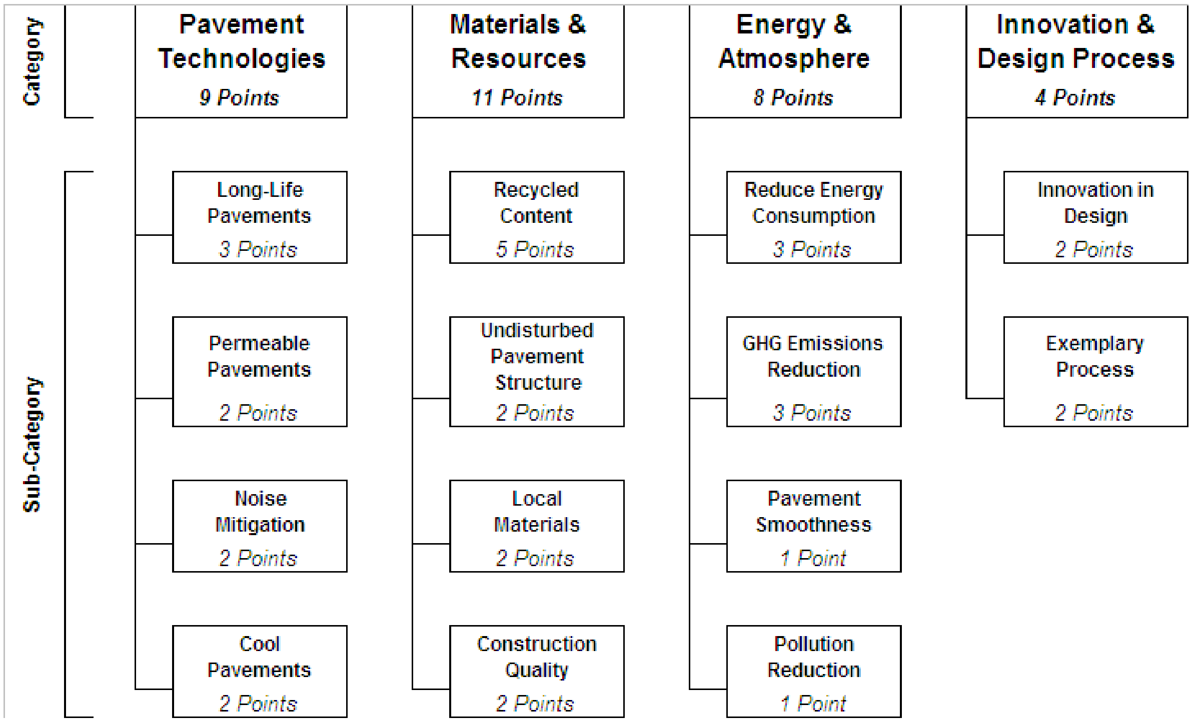


Figure 4.5.1: Overview of GreenPave Point Distribution

Specific objectives within these subcategories must be met in order to achieve the maximum points available. Proposed rating levels for the GreenPave projects are bronze (9-12 points), silver (12-15 points), gold (15+ points), and trillium (for future development). For more details, refer to the GreenPave Reference Guide [32].

Ultimately, the development of GreenPave is to enhance the sustainability of Ontario's transportation infrastructure through designing and selecting the most economical and environmental-friendly pavement treatment alternatives.

4.5.2 PRESERVATION STRATEGIES

Pavement preservation is a program employing network-level, long-term strategies that enhance pavement performance by using an integrated, cost-effective set of practices that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system, without significantly increasing the structural capacity. An effective pavement preservation program treats the pavement while it is still in good condition before it gets to the onset of serious deterioration. By applying a cost effective treatment at the right time, the pavement is restored almost to its original condition (Figure 4.5.2). The pavement will be maintained in a reasonably good condition, which will defer costly rehabilitation and reconstruction. In addition, performing a series of successive pavement preservation treatments is less disruptive to traffic than the long closures associated with rehabilitation and reconstruction contracts.

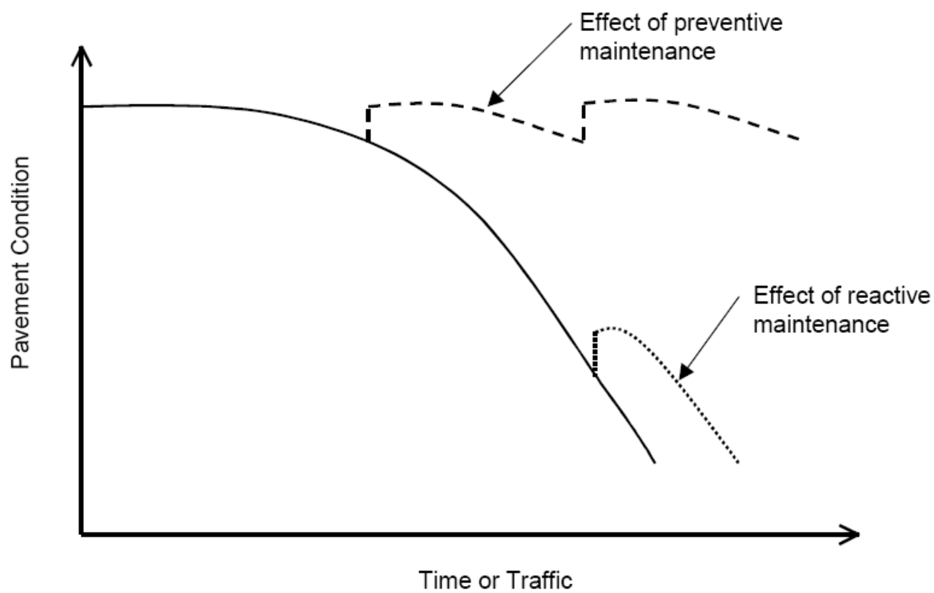


Figure 4.5.2: Applying pavement treatments at the optimal time can maintain the pavement in good condition. [33]

Pavement rehabilitation strategy has typically been a reactive treatment, which means we often apply the rehabilitation treatment when it is triggered by a pavement reaching a minimum service level. As the road gets closer to the end of its service life, it requires more expensive and time-consuming fixes in order to improve the pavement condition.

A pavement preservation program, which is significantly different from the usual way of doing business, is a cost effective concept where preventive treatments are applied during the early life of the pavement. But in some cases areas with poor subgrade or a pavement with structural damage, those should not be considered for preservation treatment. If we spend $1 now to apply preventive maintenance on the road, we can defer the $4 to $5 rehabilitation or reconstruction costs to future years and prolong the pavement life (Figure 4.5.3). Pavement preservation treatments maintain the pavement at a high level of serviceability. Preventive treatments usually involve thin treatments, relatively less time consuming operations, and minimal disruption to the traveling public. These include, seal coat (with our without stress absorbing membrane), slurry seal, micro-surfacing, chip seal/surface treatment, thin HMA overlays, crack sealing, etc... Details of the list of preservation treatments are described under section 4.2.2, 4.3.2 and 4.4.2.

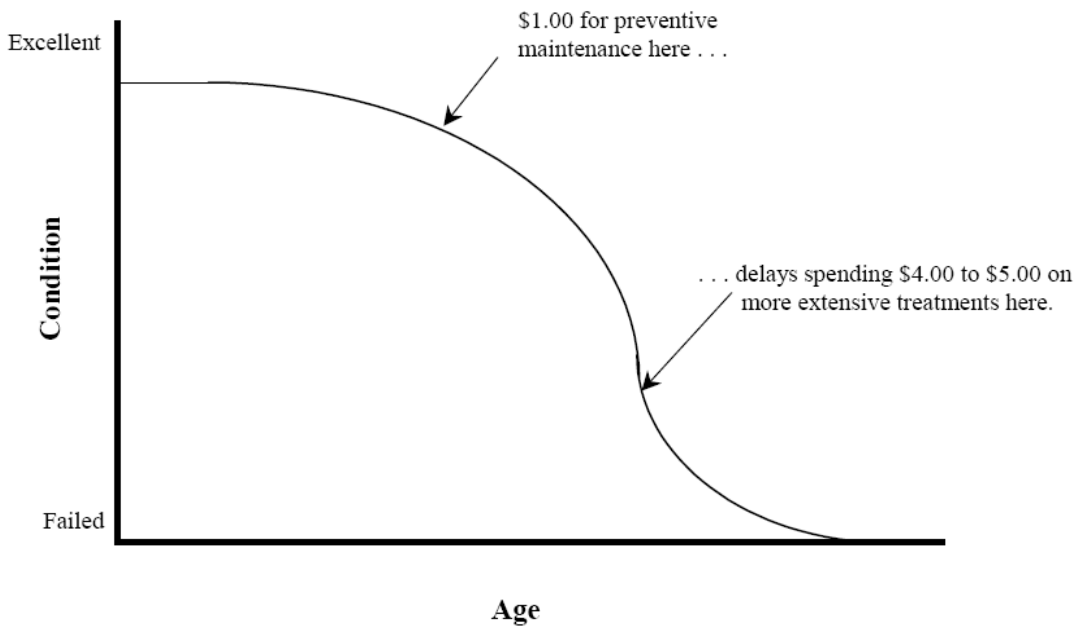


Figure 4.5.3: Applying pavement treatments at the optimal time provides the most efficient use of funds to extend the life of the pavement [33]

It is important to note that preservation strategies are not meant to replace pavement rehabilitation, but a cost effective solution to prolong pavement life. Ultimately, pavement preservation should be an integral part of the overall pavement management to support its need to ensure that preventive treatments are applied at the appropriate time. More information on pavement preservation can be found on Reference [33] and [34].

REFERENCES TO CHAPTER 4

[1] Ministry of Transportation and Communication, *Maintenance Manual: Maintenance Quality Standards (MQS) and Maintenance Best Practices (MBP)*, Highway Engineering Division, Transportation Operations Branch, Downsview, Ontario, 1992.

[2] Chong, G.J., Jewer, F.W. and Macey, K, *Pavement Maintenance Guidelines: Distresses, Maintenance Alternatives and Performance Standard*, SP-001, Ministry of Transportation and Communications, Downsview, Ontario, August 1989.

[3] Hajek, J.J., Phang, W.A., Wrong, G.A., Prakash, A. and Stott, G.M., *Pavement Condition Index (PCI) For Flexible Pavements*, PAV-86-42, Ministry of Transportation and Communications, Downsview, Ontario, August 1986.

[4] Transportation Association of Canada, *Pavement Design and Management Guide*, TAC, Ottawa Canada, 1997.

[5] Eaton, R.A., Joubert, R.H. and Wright, E.A., *Pothole Primer: A public administrator's guide to understanding and managing the pothole problem*, U.S. Army Corps of Engineers Special Report 81-21, September 1981 (revised April 1985).

[6] Strategic Highway Research Program, *Asphalt Repair Manuals of Practice*, SHRP, Washington, 1993.

[7] Lynch, D and Korgemagi, P., Evaluation *of a Mechanized Spray Patching Machine*, Ministry of Transportation and Communications, Downsview, Ontario.

[8] Chong, G.J., Phang, W.A. and Wrong, G.A., *Manual for Condition Rating of Flexible Pavements -Distress Manifestations*, SP-004, Ministry of Transportation and Communications, Downsview, Ontario, April 1982.

[9] Evers, R.C. and Lynch, D.F., *Sealing Cracks In Asphaltic Concrete Pavements*, Proceedings of the 28th Annual Conference of the Canadian Technical Asphalt Association, Quebec, November 1983.

[10] Chong, G.J., *Consequences of Deferred Maintenance On Transverse Cracks In Cold Areas*, proceeding of the 3rd Canada/Japan workshop, Paving in Cold Areas, Ottawa, July 1987.

[11] Chong, G.J., Improved Preventative Maintenance: *Sealing Cracks in Flexible Pavements in Cold Regions*, PAV-87-01, Ontario Ministry of Transportation, December 1987.

[12] Ponniah, Joseph and Gerhard Kennepohl. *The Economic Benefits of Sealing Cracks in Flexible Pavements*, Proceedings of the Canadian Technical Asphalt Association, Regina, November, 1994.

[l3] The Asphalt Institute, "*Asphalt Cold-Mix Recycling*" Manual Series No. 21 (MS-21), March 1983.

[14] Kazmierowski, T., A. Bradbury and P. Marks, "*Performance of Cold In-place Recycling in Ontario*", Research Record No. 1337, TRB, 1992.

[15] Lane, B., T. Kazmierowski and S. Chan, "*Evaluation of Cold In-Place Recycling Expanded Asphalt on Highway 7 Perth, Ontario*", Proceedings of the Canadian Technical Asphalt Association Conference, Montreal, November 2004.

[16] Kazmierowski, T.J., A. Bradbury and P. Marks, "*Seven Years of Experience with Hot In-place Recycling in Ontario*", Proceedings of the Canadian Technical Asphalt Association Conference, Regina, November 1994.

[17] Martec Recycling Corporation – Sustainable Roads.
http://www.martec.ca/media_room/files/publications/Broch01G3u8page.pdf

[18] Corkill, J.T., *Construction and Performance of Cold Mix Placed by the Midland Mix Paver*, Ministry of Transportation Report EM- 15, Downsview, Ontario, November 1977.

[19] Chong, G.J., Phang, W.A. and Wrong, G.A., *Manual for Condition Rating of Rigid Pavements*, SP-005, Ministry of Transportation, Downsview, Ontario, April 1982.

[20] American Concrete Pavement Association, "*Joint and Crack Sealing and Repair for Concrete Pavements*", TB-012.P, ACPA, 1993.

[21] Transportation Research Board, *Joint Repair Methods For Portland Cement Concrete Pavements*, NCHRP Report 281, Transportation Research Board, December 1985.

[22] Kazmierowski, T.J. and Wrong, G.A., *Experimental Cement Grout Subsealing of a Concrete Pavement in Ontario*, Presentation at the 1986 RTAC Annual Conference, Toronto, Ontario, September 1986.

[23] American Concrete Pavement Association, "*Diamond Grinding and Concrete Pavement Restoration 200*", TB-008 CPR, ACPA, 1990.

[24] Portland Cement Association, *Guide to Concrete Resurfacing Designs and Selection Criteria*, Skokie, Illinois, 1981.

[25] Portland Cement Association, *Patching Concrete Pavements*, PCA Information Leaflet, Skokie, Illinois, 1976.

[26] Sturm, H. J., Kazmierowski, T. J. et al, "Fast Track " *Concrete Pavement Repairs, Ontario's Experience*, Paper presented at the 1989 RTAC Annual Conference, Calgary, Alberta, September 1989.

[27] Lane, B. and Kazmierowski T., *Precast concrete slab repair methods : Innovative pavement repair trials in Canada*, American Concrete Institute, Concrete International 2006 Volume 28, November 2006.

[28] Alkins, A., Lane, B. and Kazmierowski T., *Sustainable Pavements – Environmental, Economic and Social Benefits of In-situ Pavement Recycling*, Transportation Research Board, Issue number 2084. 2008.

[29] Greenroads Rating System v1.0. University of Washington. 2010.

[30] GreenLITES Project Design Certification Program. New York State Department of Transportation. September 2008.

[31] Green Guide for Roads. Alberta Ministry of Transportation. Stantec, January 2008.

[32] *GreenPave Reference Guide, Version 2.0*. Ontario Ministry of Transportation, Materials Engineering and Research Office, Pavements and Foundations Section. March 2014.

[33] Foundation for Pavement Preservation (FP2)
http://www.fp2.org/

[34] National Center for Pavement Preservation (NCPP)
http://www.pavementpreservation.org/

APPENDIX A:

DECISION MATRIX FOR MAINTENANCE, PRESERVATION AND REHABILITATION TREATMENTS

A1. Decision Matrix for Flexible Pavement:

Treatment Activity		Restoring or Improving Pavement Surface in Terms of: Preventing Water Infiltration	Localized Severe Distress	Bleeding, Ravelling, or Poor Skid Resistance	Ride Quality	Environmental Deterioration	Structural Capacity & Traffic	Expected Service Life (years)
Routine Maintenance	Pothole Repair	●	◆		●			< 1
	Roadside maintenance					◆		1-5
	Drainage maintenance	●	❍			◆		2-5
	Spray patching	◆		❍	❍			2-5
	Localized distortion repair		◆		●	❍		2-5
Minor Rehabilitation / Preservation*	Rout & crack sealing*	◆						1-5
	Hot mix patching*	●		◆	❍			5-10
	Surface sealing* (seal coat, slurry seal, micro-surfacing, chip seal / surface treatment)	●		◆	●	❍		3-7
	Texturization* (micro-milling, shot blasting, sand blasting)			◆	❍	❍	❍	1-6
	Asphalt strip repair* / full depth patching	❍	◆		❍		●	5-10
	Hot mix resurfacing*	●		◆	❍	❍	●	5-12
	Partial depth removal (milling) & resurfacing*	❍	❍	●	◆	●	●	8-12
	In-place recycling* (HIR, CIR, CIREAM)			●	●	❍	◆	7-15+

Treatment Activity		Restoring or Improving Pavement Surface in Terms of:						
		Preventing Water Infiltration	Localized Severe Distress	Bleeding, Ravelling, or Poor Skid Resistance	Ride Quality	Environmental Deterioration	Structural Capacity & Traffic	Expected Service Life (years)
Minor Rehabilitation / Preservation*	Cold mix with sealing course*	❍		◆		❍		5-10
	Distortion corrections		❍		◆	❍		5-10
	Drainage improvements	●	❍			◆		7-10
	Frost treatments		◆		●	●		3-5
	Roadside slopes and erosion control	❍				◆		3-7
Major Rehabilitation	Full depth removal & resurfacing			●	◆	❍	●	8-12
	Full depth reclamation / pulverization			❍	●	●	◆	12-15+
	Pulverization with expanded asphalt stabilization			❍	●	●	◆	12-15+
	Whitetopping			❍	❍	❍	◆	5-10
	Unbonded Concrete Overlays			❍	❍	❍	◆	25+

Legend:

* Pavement Preservation Treatments
◆ Primary Application
● Commonly Used
❍ May be considered

A.2. Decision Matrix for Rigid Pavement:

Treatment Activity		Restoring or Improving Pavement Surface in Terms of:						Expected Service Life (years)
		Preventing Water Infiltration/ Incompressibles	**Localized Severe Distress**	**Faulting, Spalling, or Poor Skid Resistance**	**Ride Quality**	**Environmental Deterioration**	**Structural Capacity & Traffic**	
Routine Maintenance	Pothole Repair	●	◆		●			< 1
Routine Maintenance	Spall Repairs	❍	●	◆	●			3-5
Routine Maintenance	Blow ups		◆		●			1-5
Routine Maintenance	Localized distortion repair		◆			❍		2-5
Minor Rehabilitation / Preservation*	Resealing and sealing of joints and cracks*	◆						7-10
Minor Rehabilitation / Preservation*	Load transfer retrofit*			◆	●		❍	10-15
Minor Rehabilitation / Preservation*	Full depth joint repair*	❍		◆	●		❍	10-15
Minor Rehabilitation / Preservation*	Full depth stress relief joints*			◆		●		5-10
Minor Rehabilitation / Preservation*	Milling of stepped joints and distortion*			●	◆	❍		8-10
Minor Rehabilitation / Preservation*	Subsealing & joint stabilization	●	❍	●				5-10
Minor Rehabilitation / Preservation*	Surface texturization / diamond grinding*			◆	●			8-12
Minor Rehabilitation / Preservation*	HMA resurfacing*	●		❍	●	❍	❍	5-10

Treatment Activity		Restoring or Improving Pavement Surface in Terms of:						Expected Service Life (years)
		Preventing Water Infiltration/ Incompressibles	**Localized Severe Distress**	**Faulting, Spalling, or Poor Skid Resistance**	**Ride Quality**	**Environmental Deterioration**	**Structural Capacity & Traffic**	
Major Rehabilitation	Bonded Concrete Overlays			❍	❍		●	6-10+
	Unbonded Concrete Overlays			❍	❍	❍	◆	25+
	Crack & Seat and Resurfacing			●	❍		◆	10-15
	Rubblization and Resurfacing			●	●	❍	◆	10-15
	Widening and shoulder retrofits	◆				❍		5-10
	Full depth slab repair*		◆				●	5-15
	Precast concrete slab repair*		◆				❍	5-15

Legend:

- * Pavement Preservation Treatments
- ◆ Primary Application
- ● Commonly Used
- ❍ May be considered

A.3. Decision Matrix for Surface Treatment Pavement:

Treatment Activity		Restoring or Improving Pavement Surface in Terms of:						Expected Service Life (Years)
		Preventing Water Infiltration	Localized Severe Distress	Bleeding, Raveling, Skid	Ride Quality	Environmental Deterioration	Structural Capacity & Traffic	
Routine Maintenance	Drainage Improvement	●	❍			●		1-5
Minor Rehabilitation / Preservations*	Spray Patching*	◆	●	❍	❍			2-5
	Chip Sealing*	❍	●	◆		●		3-5
	Levelling		●		●		●	2-5
	Full Depth Patching		◆		●		●	5-10
Major Rehabilitation	Surface Treatment* (single)	❍		❍	●	❍	●	2-5
	Surface Treatment* (double)	❍		❍	●	❍	●	5-9
	Pulverization and Resurfacing		❍	❍	◆	●	●	5-10

Legend:

* Pavement Preservation Treatments
◆ Primary Application
● Commonly Used
❍ May be considered

Chapter 5:

EVALUATION AND PAVEMENT MANAGEMENT

(Blank Page)

5 Evaluation and Pavement Management

This chapter outlines the methods of evaluating the characteristics and performance of pavements. Second, it presents the manner in which these evaluations are used for pavement management.

5.1 PAVEMENT EVALUATION AND PERFORMANCE

5.1.1 PURPOSE OF EVALUATION

Pavement evaluation refers to the procedure of field measurement and/or observing the current state of various pavement characteristics and documents them for future use. Both objective and subjective performance data are used to indicate: how well the pavement serves the travelling public at the present time; how the condition of the pavement has changed over time; whether the pavement is in need of repair and the extent of repair required.

The need for pavement evaluation:

- Provide input for calculating the pavement performance;
- Provide input to pavement design;
- Check the pavement conditions to ensure level of service and to trigger preventive and/or corrective treatments;
- Provide input to calculate funding allocation;
- Support the efforts of research and development.

Below are the four different kinds of pavement evaluation surveys conducted by MTO or consultants. Details of these surveys to be discussed in the following sub-sections:

- Pavement distress

- Pavement roughness
- Pavement transverse profile and texture
- Pavement structure

5.1.2 EVALUATION OF DISTRESSES

The most common form of pavement evaluation is to measure pavement distresses. Pavement distress evaluation has been carried out as a routine assessment since the mid-1960s, which consists of a periodic survey of the provincial highway network. The distresses were observed, rated and subjectively combined with the ride quality to produce the Pavement Condition Rating (PCR), which was used by the ministry for evaluating pavement performance [1], [2], [3], [4], [5], [6], [7]. In 1985, the pavement condition evaluation became more objective by combining pavement distress evaluation data with mechanical measurement of surface roughness. Although distresses are still assessed using similar survey procedures as before, a new condensed format [8] was developed to record the results of the survey for calculating the Pavement Condition Index (PCI). Refer to Appendix A and B for details.

Summary of Procedures

The following procedure is used to measure pavement distress in order to determine PCR or PCI. Summary of the procedures are presented in References [2], [3], [4] and [5]:

Step 1: Drive over pavement at posted speed and establish subsections to be rated individually.

Step 2: Drive over subsection at a speed suitable for observation. Make at least one stop (depends on the amount of distress) to examine distress type, density and severity.

Step 3: Record the results on the Pavement Evaluation Form, shown in Appendix A. This is done using the standard classification. The tables on Appendix B are the severity and density for evaluating different pavement types.

5.1.3 EVALUATION OF ROUGHNESS

The second component of pavement evaluation consists of measuring pavement roughness. Roughness is an indication of the surface irregularities which impacts the ride quality. The PCR survey subjectively assigned roughness value in the form of a Ride Condition Rating (RCR). Since 1985, emphasis has been given to select a mechanical roughness measuring device to calculate PCI.

There are many reasons for measuring roughness. The most prominent one is that roughness is a good indication of level of service to the travelling public. In fact, studies have shown that the cost of operating a vehicle is affected by roughness. Hence, roughness plays an important role in pavement management decisions.

MTO expresses roughness by correlating all mechanical measures to the RCR rated by the panel. In general, there are three methods for measuring roughness, each of which has its own measurement procedures and/or instrument(s). Depending on the circumstances, all three methods are used in Ontario.

Subjective panel rating survey where a group of people or an individual drives over the road and assigns the RCR based on their subjective assessment. This method is used for the annual pavement condition survey and for researching the correlation results obtained from the mechanical methods discussed below.

Objective survey uses a mechanical measurement device to assess pavement roughness. Automated Road Analyzer (ARAN®) unit is used to determine PCI for pavement management purposes and to evaluate pavement roughness on special requests. Figure 5.1.1 below is the sample roughness survey request form.

Profile type surveys measure the relative longitudinal profile of the pavement surface along the length of the road. This method is used as an acceptance tool for specifications in concrete pavements.

REGION ____________ ARAN ☐

REQUESTED BY:

QUALITY ASSURANCE ☐

GEOTECHNICAL ☐

L.H.R.S	DIST.	HWY.	LOCATION	LENGTH (km)	NEW CONST.		RESURFACING		COMPLETION DATE FOR NEW CONST./ RESURFACING	CARRYOVER		REMARKS
					Before	After	Before	After		Yes	No	

Figure 5.1.1 Sample Roughness Survey Request Form

Riding Condition Rating (RCR)

The RCR is a subjective roughness rating that is assigned to a road section by individual raters. It is one of the surveys required to determine PCI. References [2], [3], [4] and [5] explain the procedure to assign the RCR for the PCR survey. In short, the procedure is shown below:

Step 1: Drive over the road section at the posted speed;

Step 2: Classify the pavement's riding condition and assign the RCR by selecting a number associated with the description presented in References [2], [3], [4] and [5].

International Roughness Index (IRI)

International Roughness Index (IRI) is the pavement roughness measurement standard and analysis. It is a roughness indication that is valid for any road surface type and covers all levels of roughness. An IRI value of 0 mm/m (or m/km) indicates absolute smoothness. An IRI value in the order of 10 mm/m (m/km) represents a rough unpaved roadway.

MTO measures IRI using the Automated Road Analyzer (ARAN®) which is discussed in details in the following subsection. Before using ARAN®, pavement roughness was measured using PURD (Portable Universal Roughness Device). The transfer function was developed at the time of switching from PURD-based measurements to IRI (m/km) measurements. Following is the established relationship between Ride Comfort Rating (RCR, scale of 0 to 10) and IRI for asphalt concrete surface and concrete pavement surface [9]:

For asphalt pavements:

$$RCR = 8.52 - 7.49 * \log_{10}(IRI)$$

For concrete pavements:

$$RCR = 9.27 - 6.22 * \log_{10}(IRI)$$

Automated Road Analyzer (ARAN®)

The ARAN® vehicle (Figure 5.1.2) is an objective automated system for collecting pavement performance indices, such as roughness, rutting, crossfall, digital video log and GPS (Global Positioning System).

Figure 5.1.2 Automated Road Analyzer (ARAN®)

The ARAN® device is a high speed profiler to monitor the quality of roads. A continually adaptive technology, ARAN's capabilities currently include HD imagery, position and orientation system/land vehicle (POS LV), Laser Crack Measurement System (LCMS), and RoLine Laser Profilometer. These systems can be used to compose image catalogues, GIS integration, analyze wheel rutting and road roughness to obtain International Roughness Index (IRI). The determined roughness will give an indication of ride quality and may indicate the need for maintenance or rehabilitation.

Roughness: The ARAN® uses RoLine Laser Profilometer System, which offers an advantage over single point or multi-point methods by providing a 100 mm line of data across the road surface. By seeing more of the road, RoLine allows profilers to measure accurately on all pavement types at any speed. Measurements taken with the ARAN® are converted to an accurate real-time international roughness index (IRI). These calculations can be made at a speed up to a maximum of 100km/h and as low as 22km/h. The IRI from an ARAN® survey is primarily used in the calculation of the pavement PCI.

Rutting: Located on the rear of the ARAN® is a Laser Crack Measurement System (LCMS) for crack detection and severity that scan a 4 m pavement width with a 1 mm transverse resolution, 4160 point rutting, multiple macro-texture measurements, as well as 3D and 2D data to characterize potholes, patching, raveling, sealed cracks, joints in concrete, tinning, etc. This data is in numerical form during collection but is processed in graphical or tabular format.

Global Positioning System (GPS): The GPS system is used to provide the location and coordinates of road features and assets that can be marked by the operator (i.e.; railroad crossings, intersections and bridge decks) so that the collected data can be integrated into a Geographic Information System (GIS).

Position and Orientation System for Land Vehicles (POS LV™): The POS LV™ system utilizes an onboard Inertial Measuring Unit (IMU), as well as the Distance Measuring Instrument (DMI) and the onboard GPS system, to provide accurate position data < 1m in optimal conditions. This system gives us accurate crossfall data (slope of the road); and also corrects for GPS loss by either interpolating/extrapolating (e.g.: due to tunnels, tree canopies, bridges or rock cuts). It also provides accurate location of videolog images taken at 5 meters intervals along the surveyed portion of roadway.

Distance Measuring Instrument (DMI): Quite possibly the most important of all the ARAN® subsystems, the DMI is a pulse-based system, which uses an optical shaft encoder that is attached to the rear wheel of the ARAN®. This instrument allows the ARAN® to collect data based on pulses, thus allowing the ARAN® to collect at various speeds without loss of data integrity.

High Definition Right of Way Video Camera (HD ROW): The ARAN® collects a high definition image of the road every 5 meters. The combination of 2 cameras allows for an approximate 160 degree panoramic view of the roadway. These images can be used for various applications, including pavement, sign, or barrier conditions. Figure 5.1.3 is a sample image collected using HD ROW.

Figure 5.1.3 Sample HD ROW Image

California Profilograph

The California Profilograph (Figure 5.1.4) is a profile measuring device which is used for acceptance testing of new asphalt and concrete pavements. Profile measurement is also an indication of pavement roughness.

A profilograph consists of a frame, 7.62 m long, supported by a set of independent bogey wheels at each end and with a measuring wheel in the middle as shown below:

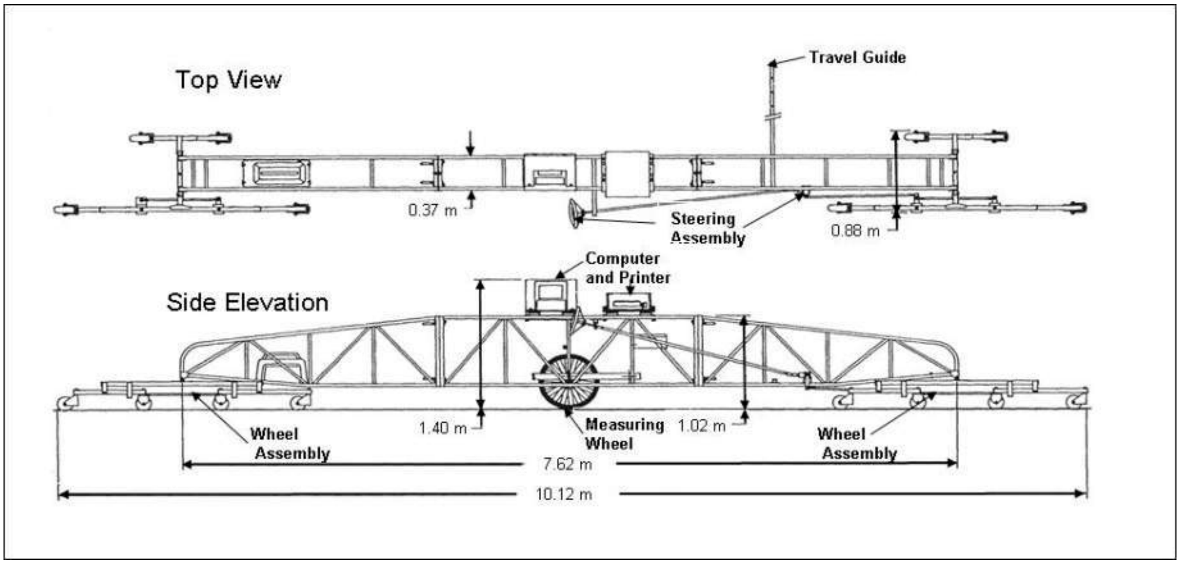


Figure 5.1.4 California Profilograph

The bogey wheels establish a datum and the measuring wheel is used to determine the distance from the pavement surface to the datum as the device is moved along the pavement section. The instrument records and prints out the pavement profile trace along each wheel path. The printed trace is then used to calculate the profile index. The profile index is determined by first summing up the amplitudes that are respectively, greater than 0.8 mm for asphalt pavements and 5.8 mm for concrete pavements of all the bumps and dips that are at least 0.6 m long along the profile length. That sum of amplitudes is then divided by the sublot length (usually 100 m) to determine the profile index in mm/km.

5.1.4 EVALUATION OF TRANSVERSE PROFILE AND TEXTURE

The surface characteristics of a pavement are measured in terms of skid resistance and rut depth. The devices and procedures used in each of these types of surveys are presented in this section.

Skid Resistance

Skid resistance is expressed as the frictional resistance between a wet pavement and the locked tires of a moving vehicle. It is related to the exposed aggregate on the wearing surface in terms of:

(a) Macrotexture - its ability to channel water away from under the tire, and
(b) Microtexture - how angular it is.

The relative skid resistance of a pavement is given in terms of a friction number (FN). This number ranges from 0 (worst) to 100 (best) and is obtained by field measurement and correlation.

Coefficient of friction, (μ) is calculated by dividing the frictional resistance to motion in the plane of the interface, (F) by the load acting perpendicular to the interface, (L). It is dependent on the contact area and therefore not a wholly suitable representation of the actual pairing that occurs between the tire and the pavement surface (ie, frictional resistance is not simply a pavement property).

Various conditions supplementary to the pavement itself influence the rolling, slipping or skidding of a tire, particularly when water is present. The preferred or usually used term is “friction factor”, (f), which is calculated as follows:

$$\mu = f = F/L$$

where: f = friction factor
F = frictional resistance force in the direction of travel
L = vertical load

The most widely used friction measurement standard is described in ASTM Method E274. Friction measurements made according to this method are termed “Friction Number”, (FN) and are calculated as follows:

$$FN = 100 * f = 100 * F/L$$

The skid resistance of a section is not measured on a regular annual survey. To request an

ASTM E 274 survey, the form shown in Figure 5.1.5 must be completed and sent to the Pavements and Foundations Section. When the skid resistance is required, an ASTM E 274 brakeforce unit is used. ASTM E 501 test tires are utilized on the trailer portion of the tester.

FRICTION SURVEY REQUEST

Location

BEGINNING- L.H.R.S.: ______ Offset: ______ km District #: ______

END- L.H.R.S.: ______ Offset: ______ km HWY.#: ______

SITE LENGTH: ______ km *DIRECTON(S)-* ______

TOTAL NO. OF LANES- ______

Reason For Investigation

- ☐ Accident Inquiry
- ☐ Special Request
- ☐ Spot Hazard
- ☐ Project Request
- ☐ Surveillance

Comments- Contract No., Pavement Type, Details

Location Description

Traffic Flow

- ☐ Constant Speed
- ☐ Acceleration
- ☐ Deceleraton
- ☐ Cornering
- ☐ Intersection
- ☐ Traffic Signals
- ☐ Entrances/Exits

Road Character

- ☐ Controlled Access
- ☐ Uncontrolled Access
- ☐ Collector Lanes
- ☐ Core Lanes
- ☐ Transfer Lanes
- ☐ On Ramp
- ☐ Off Ramp

Surf. Condition

- ☐ Flushing
- ☐ C/A Loss
- ☐ C/A Polishing
- ☐ Cracking
- ☐ Ravelling
- ☐ Wheel Track Rutting
- ☐ Patching
- ☐ Contamination

Accident Data

Year	Wet & Dry Accidents	Wet Accidents	% Wet

REQUESTED BY: ______ OFFICE: ______

DATE OF REQUEST: ______ PHONE: ______

FRANK MARCIELLO- SUPERVISOR: PAVEMENT EVALUATION UNIT

Figure 5.1.5 Skid Resistance Test Request Form

ASTM E 274 Brake Force Trailer

The Brake Force Trailer (Figure 5.1.6) is equipped with a water supply, a locking wheel and transducers which measure the horizontal and vertical force required to pull the locked wheel over the wet pavement.

Figure 5.1.6 ASTM Brake Force Trailer

The pavement immediately in front of the locked wheel is sprayed with water at a rate which ensures a constant thickness of water film. The measuring wheel is locked and the weighted trailer is pulled along the pavement section. The friction number is calculated as given in Reference [10]. The friction number is the horizontal force applied to the test tire at the tire-pavement contact patch divided by the dynamic vertical load on the test wheel. It is reported at a given test speed related to the posted speed of the highway, i.e. FN80 at 80 km/h.

Photo-Interpretation Method

The photo-interpretation method consists of taking stereo photographs of the pavement surface using special equipment. The photographs are then viewed under a stereoscope. Six texture parameter numbers are noted and averaged for ten randomly selected areas on the picture. These six texture parameter numbers characterize the texture by:

1. height of projections;
2. width of projections;
3. distribution of projections;
4. angularity of projections;
5. harshness of projection's surface, and;
6. harshness of pavement surface between projections.

The six numbers are then used with charts found in Reference [11] which have been developed by correlation with the ASTM Brake Force Trailer results.

Recently, automated laser-based devices have been developed to determine pavement micro and macro texture. Microtexture is usually defined as surface irregularities of the individual components of the pavement surface such as aggregate and binder whereas macrotexture involves longer wavelengths and reflects aggregate size and distribution.

Rut Depth

Pavement surface ruts can pose a major safety concern. Ruts affect the handling of a vehicle and increase the possibility of hydroplaning on wet pavements. No absolute standards exist for relating excessive ruts to safety factor.

Ruts are categorized as either traffic load associated deformation, wear related, or a combination of the two. Rut depth is defined as the depth of the depressions in the wheel path. This depth is reported as the distance from a reference to the pavement surface in the wheel path. Measurement of pavement ruts can vary from simple visual assessments to automated techniques that use lasers or ultrasound techniques to measure transverse pavement profiles at full highway speeds. The three alternate methods of defining the reference for measuring the rut depth are: long bar, short bar and string. The measurements for each of these are illustrated in Figure 5.1.7. The depth of the ruts will be different depending on the type of measurement.

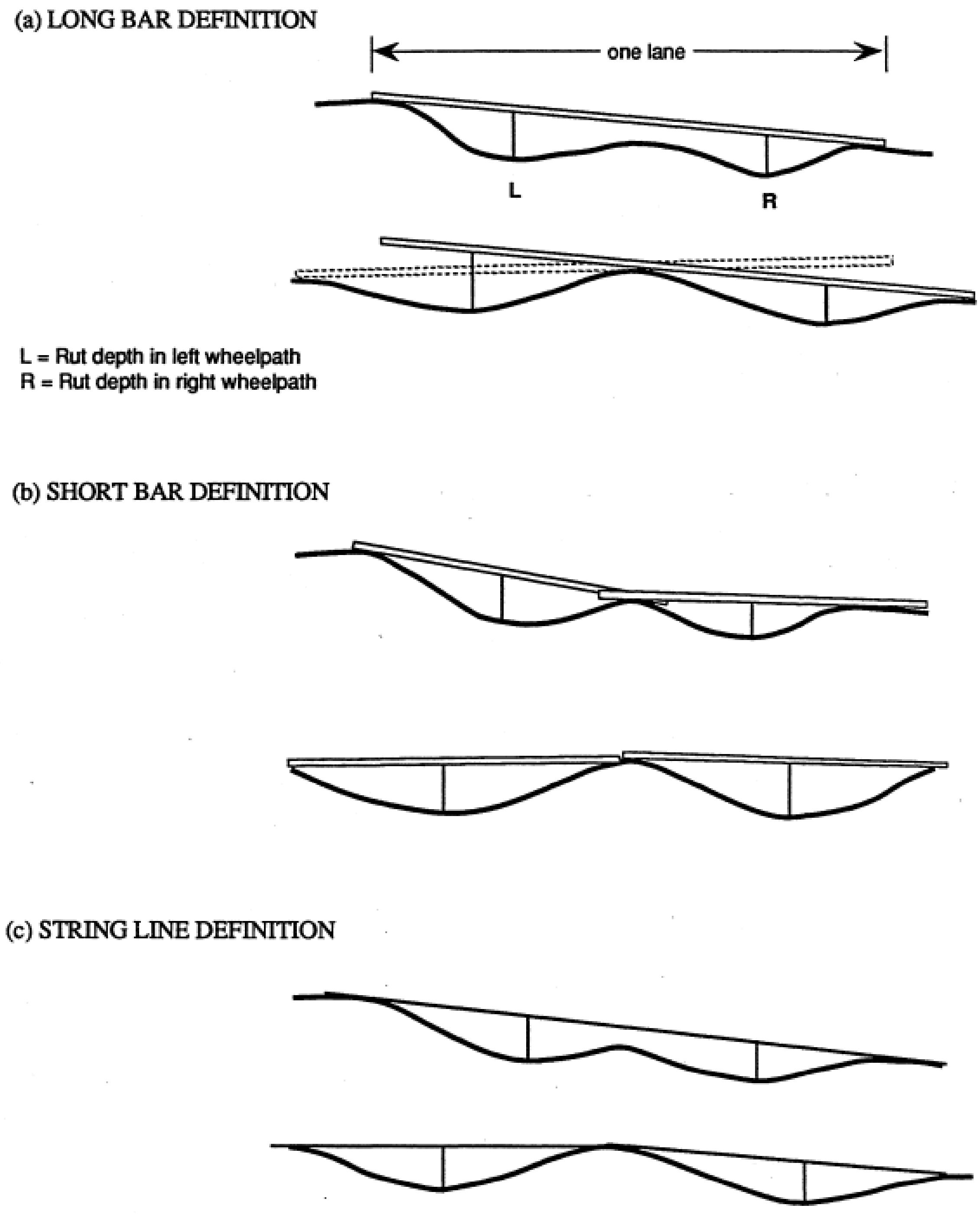


Figure 5.1.7 Alternative Methods of Measuring Rut Depth

The instability rutting of asphalt takes place because asphalt concrete exhibits lower stiffness particularly under slow-moving or standing loads. Instability asphalt rutting is aggravated at higher pavement temperatures, since the stiffness of asphalt concrete is further decreased at higher asphalt pavement temperatures.

Another shape of rut has occurred in flexible pavements. The smooth flowing depression in each wheel path has changed to a double rut [12]. This appears to be a result of higher tire pressures associated with dual radial tires, increased truck volumes and higher axle loadings. Instability of the asphalt also contributed to the change in rut shape.

The rut depth is subjectively assessed as one of the distress manifestations in the cyclical distress survey. The reference required for this is the short bar. When ruts get to such a depth that they require monitoring, more precise surveys are carried out. In this case, one of four methods is available, depending on the circumstance. These are explained below.

Template Method

The template method of measuring ruts on the road is the old "stand-by" rut depth measurement. Two types of template surveys are used. The first type uses the long bar reference and involves placing a 3.75 m (12 ft) flat bar on stands across one lane. The stands (feet) ensure the rut bar need not be moved to measure rut depth in each wheel path. The maximum distance between the bar and the surface is located and measured. The rut depth is calculated by subtracting the height of the stand from the measured distance. The second form of template survey is used for the distress survey. In this method, the rut depth is measured as the distance between the surface and the short bar reference. The reference is established with a 1.25 m (4-ft) bar laid across the wheel path.

To get an appreciation of the rut depth of an entire section, these procedures are repeated at regularly-spaced intervals and a mean rut depth is calculated. These methods should only be used in places where the procedure will not endanger the survey crew or the traffic.

Rod and Level

The rod and level is the slowest, but most accurate method of measuring rut depth. With a rod and level the survey crew measures the cross-section profile of the pavement at regular intervals along the section. This method should only be used in localized instances and in places where the procedure will not endanger the survey crew or the traffic. The rod and level can simulate rut depths for all three reference methods.

ARAN®

The Automated Road Analyzer (ARAN®) uses the Laser Crack Measurement System (LCMS) to detect rutting. An example of the display from one of these surveys is shown in Figure 5.1.8. The ARAN® unit is ideal for measuring rut depths over long distances or on freeways and other roads where the traffic is a threat to the survey crew. Refer to section 5.1.3 for more descriptions of ARAN®.

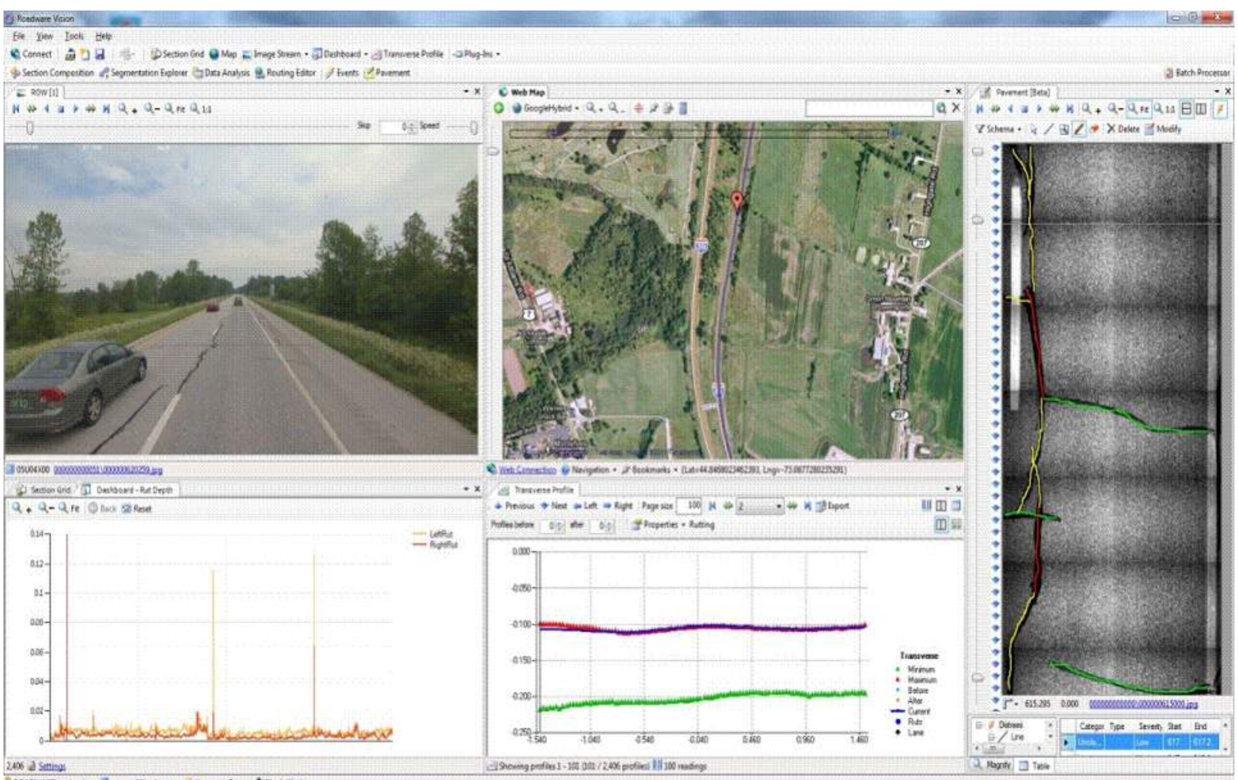


Figure 5.1.8 Sample Output from the ARAN® Rut Depth and IRI Surveys

Laser Road Surface Tester (LRST)

The Laser Road Surface Tester is a unit used for measuring rut depths that is quite similar to the ARAN®. The major difference between them is the use of laser technology rather than ultrasound. The LRST also has a "rut bar" mounted on the front bumper of a truck. This 2.5 m (8-ft) bar has laser units and light-sensitive displacement sensors spaced evenly across its length. The survey vehicle is driven over the road section at normal highway speed (30-90 km/h) and stores the rut depth readings for later processing using any one of the three reference methods.

5.1.5 EVALUATION OF PAVEMENT STRUCTURE

To design a rehabilitation treatment, the designer requires an assessment of the pavement structure and its load carrying capacity. This is achieved by performing a detailed visual survey of the road and identifying the localized distresses. Then, the designer further evaluates these areas using various investigation methods. Investigations are divided into two broad categories, destructive and non-destructive.

One of the most basic methods of evaluating the structural adequacy of a pavement is the Granular Base Equivalency (GBE) approach. The GBE expresses the contribution of each pavement components in terms of an equivalent thickness of crushed granular base. To use the GBE methodology, information on the type and thickness of the individual pavement components is required. Typically it is done through coring or probe holes. A recent technology called Ground Penetrating Radar (GPR) can detect the pavement structure thicknesses using the electromagnetic radiation. The pavement structure components are then mathematically transformed into an equivalent thickness of granular base and compared with the recommended minimum acceptable thickness to accommodate the design traffic. Additional details on the GBE procedure are given in Chapter 3.

Destructive Structural Evaluation

The most common forms of destructive pavement structure evaluations are augering, coring and cutting.

Augering

Augering is the most common technique used for investigation of pavement structure distress. A series of boreholes are augered in the immediate area of the distress, usually at pavement edge. A cross-sectional view of the entire pavement structure and subgrade can be formed using this method. The engineer measures the thickness, determines the composition of each pavement layer and identifies the causes of the distress.

Coring

Coring is commonly used in pre-contract investigations for partial and full-depth recycling projects. Coring is also used to evaluate the thickness and composition of the various pavement components. Reference [13] gives a more detailed description of the coring steps and procedures.

Coring involves drilling holes through the pavement at preselected locations. Material samples obtained from coring are used in laboratory tests which determine the existing asphalt cement content, the aggregate gradation, the recovered penetration and any evidence of stripping. The results of a coring test are recorded on an Asphalt Core Analysis sheet as shown in Figure 5.1.9. This completed sheet is included in the contract package.

To select the location of the individual cores for testing, the evaluator should use the random sampling technique described in Reference [13], unless a localized distress is being investigated.

LOCATION		CORE DATA		EXTRACTION TEST RESULTS		
STATION	OFFSET (m)	HOT MIX DEPTH (mm)	DEPTH TESTED (mm)	A.C. CONTENT	% RETAINED 4.75 mm	% PASS 75 μm

Figure 5.1.9 Sample Asphalt Core Analysis Sheet

<u>Cutting</u>

Cutting provides a cross-sectional view of the entire pavement structure down to the subgrade. Using this method, the engineer can measure the thickness, determine the composition of each pavement layer and identify the nature and location of individual distresses. Cutting is used predominantly for evaluating rutting problems and identifying the internal pavement failure mechanisms.

Non-Destructive Structural Evaluation

The two most common forms of non-destructive pavement structure evaluations are Dynaflect, and Falling Weight Deflectometer (FWD). These devices use different techniques to measure the surface deflection of the pavement under a given load. The designer uses these deflection measurements for many purposes, including pavement thickness design as detailed in Section 3.3.1. Reference [14] shows the design charts for Dynaflect.

When designing a pavement, the thickness for the various layers must be determined based on the maximum deflection desired or allowed. Deflection testing is ideally carried out in the spring when frost action and seasonal moisture fluctuations produce minimum pavement strength and maximum deflections. However, for the FWD testing, the testing procedure could damage the weak pavement structure; therefore it is not recommended to perform FWD testing in the spring.

In addition to the seasonal variations, the deflection of a pavement section may vary along its length. To reduce the effect of both variables, a statistical process is used when estimating the maximum deflections. The following steps are required to carry out this statistical process:

Step 1: Divide the pavement into sections which are 300 m long. The sections can be of variable length provided they are homogeneous with respect to the factors which influence the pavement design (e.g., subgrade and traffic);

Step 2: Select at least 10 test points within each section using a table of random numbers. Or select more than 10 points which are equidistant from each other,

Step 3: Obtain the deflection measurement at the selected test points;

Step 4: Calculate the average deflection ($\overline{X}$) and the standard deviation (σx).

Step 5: Calculate the estimated maximum probable deflection value using this equation:

$$D\max = (\overline{X} + 2\sigma x)$$

Dynaflect

Dynaflect uses a dynamic cyclical force generator mounted on a trailer to apply a dynamic load. Dynaflect is the least expensive and is relatively simple to operate automated pavement deflection measuring device. In comparison, the Dynatest has been shown to have a low coefficient of variation with excellent repeatability. It is a trailer-mounted system and measures deflections procedure by impulses generated by a pair of unbalanced flywheels rotating in opposite directions. Load pulses are transmitted to the pavement through a pair of rigid wheels. A series of geophones at predetermined locations measures the pavement surface deflection under a harmonic constant load level of about 4kN. The deflection at five locations is measured with geophones suspended from the towing arm of the trailer and these measurements are plotted to illustrate the deflection basin. A schematic of the Dynaflect is shown in Figure 5.1.10.

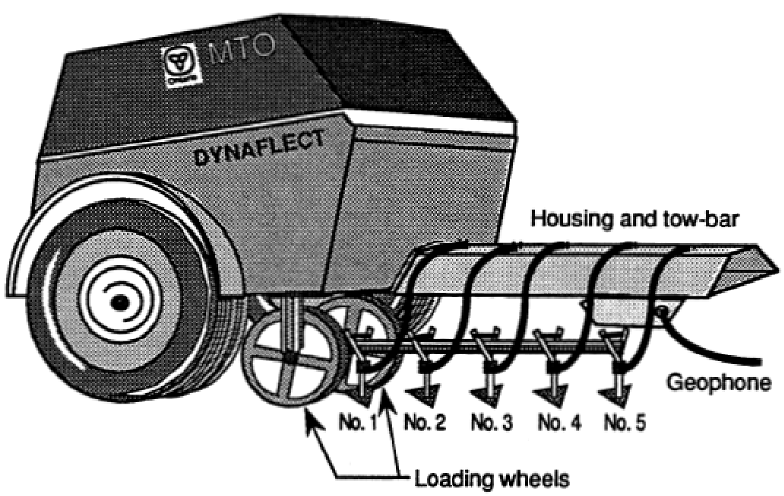


Figure 5.1.10 Dynaflect

The steps to perform Dynaflect test are as follows:

Step 1: Select the test points as explained above;

Step 2: Position the Dynaflect over the test point and lower the steel wheels to contact the pavement surface;

Step 3: Lower the geophones to contact the pavement surface;

Step 4: Apply the dynamic force to the pavement via the steel wheels;

Step 5: Record the deflection data;

Step 6: Raise the steel wheel and the geophones.

Falling Weight Deflectometer (FWD)

Falling Weight Deflectometer (FWD) is a trailer-mounted, computerized non-destructive testing device which imparts a dynamic load impulse to the pavement surface by dropping a mass from some specific height. By varying the height of fall and/or drop mass, the peak force applied to the pavement can be varied. An impulse load is applied to the loading plate by dropping a weight package on a dampening system, with the resulting pavement deflection measured by seven seismic transducers (geophones) spaced at predetermined intervals from the loading plate. A microcomputer monitors and controls the complete operations of the FWD from the tow vehicle with the test data stored for later processing. Loads applied to the pavement range from about 5 to 250 kN with a loading pulse in the range of 25 to 30 milliseconds. The impact force of the weight simulates the actual load applied by a moving vehicle. See Reference [15] for more details on FWD testing procedures.

The surface deflections are measured by several transducers and recorded for subsequent analysis. A typical FWD is shown in Figure 5.1.11:

Figure 5.1.11 Falling Weight Deflectometer

Also, a portable FWD unit is available to measure pavement structural adequacy in the field. However, the calibration and data analysis requirement for the portable FWD is not the same as the traditional trailer-mounted FWD. Data correlation analysis is required when comparing the results produced from the two units.

5.1.6 EVALUATION OF PAVEMENT PERFORMANCE

Two types of pavement evaluations are carried out to establish pavement performance. These evaluations are:

1. Pavement roughness which indicates the ability of the pavement to serve the travelling public.
2. Pavement distress which indicates the need, type and timing for repair.

Pavement performance has been systematically measured since the mid- 1960s using the Pavement Condition Rating (PCR). The PCR was originally developed as a subjective measure to assist in rehabilitation scheduling. Since its development, the PCR has been used as an inventory of structure, a history of construction, to allocate budgets and as a feedback measure for pavement design.

The Pavement Condition Index (PCI) was developed for MTO to provide a more objective and reliable measure of pavement performance. Replacing the PCR with the PCI will help establish a reproducible and acceptable measure of pavement performance. The PCR is described in Reference [2] for flexible, [3] for rigid, and [4] for surface treated pavements. The PCI is described in Reference [8].

The serviceability of a pavement is represented by a plot of its performance in terms of PCI or PCR versus time or accumulated traffic, as shown in Figure 5.1.12. Two pavements with different structures are indicated in the figure. Pavement A, with a lower strength, reaches its terminal performance level before Pavement B. Pavement A applied the first rehabilitation treatment and extended the life of the pavement. After the treatment is applied to Pavement A, the performance level is expected to rise approximately to its original value.

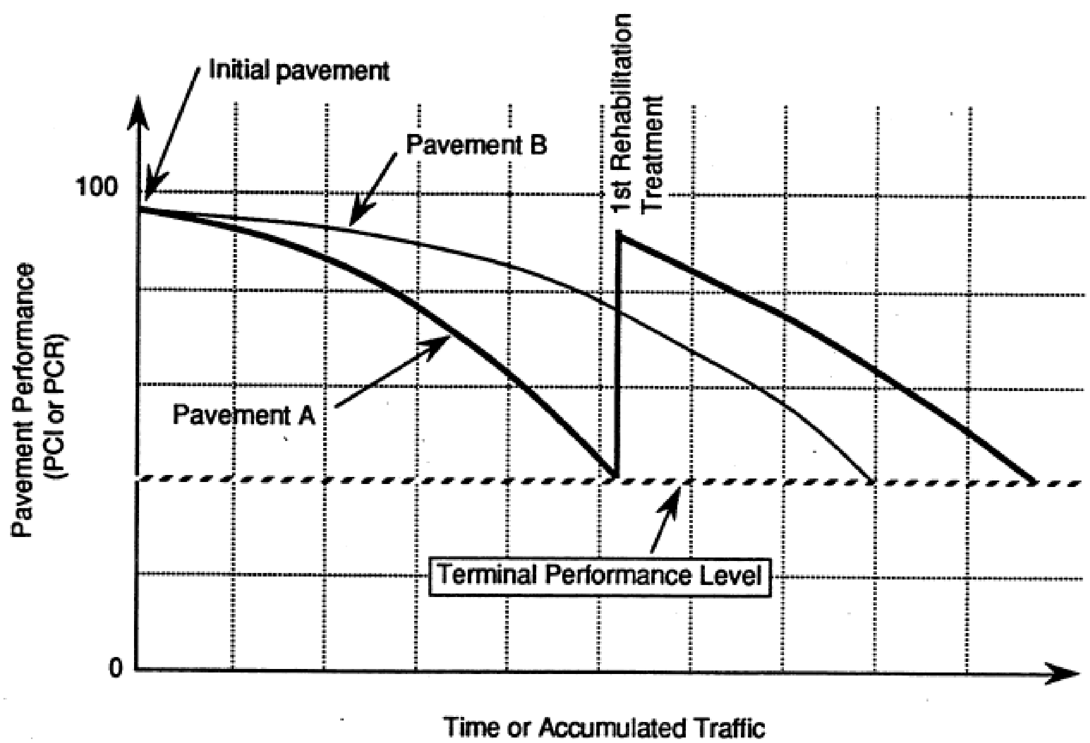


Figure 5.1.12 Pavement Performance Concept

Pavement Condition Rating (PCR)

PCR is assigned to a pavement section by an individual rater. The rater follows the procedure given in References [2], [3] or [4] to evaluate the roughness and distress of the pavement. In consideration of these two parameters, the rater assigns a PCR value in accordance with the following descriptions (Table 5.1.1):

Table 5.1.1 PCR Description of Pavement Section

PCR	Description of Pavement Section
0 – 20	Pavement is in poor to very poor condition with extensive severe cracking, alligatoring and dishing. Rideability is poor and the surface is very rough and uneven.
20 – 30	Pavement is in poor condition with moderate alligatoring and extensive severe cracking and dishing. Rideability is poor and the surface is very rough and uneven.
30 – 40	Pavement is in poor to fair condition with frequent moderate alligatoring and extensive moderate cracking and dishing. Rideability is poor to fair and surface is moderately rough and uneven.
40 – 50	Pavement is in poor to fair condition with frequent moderate cracking and dishing, and intermittent moderate alligatoring. Rideability is poor to fair and surface is moderately rough and uneven.
50 – 65	Pavement is in fair condition with intermittent moderate and frequent slight cracking, and with intermittent slight or moderate alligatoring and dishing. Rideability is fair and surface is slightly rough and uneven.
65 – 75	Pavement is in fairly good condition with slight or very slight dishing and a few areas of slight alligatoring. Rideability is fairly good with intermittent rough and uneven sections.
75 – 90	Pavement is in good condition with frequent very slight or slight cracking. Rideability is good with intermittent rough and uneven sections.
90 – 100	Pavement is in excellent condition with few cracks. Rideability is excellent with few areas of slight distortion.

Pavement Condition Index (PCI)

The PCI is calculated and assigned to a pavement section objectively. The rater follows the same procedure as the pavement condition survey to evaluate and collect the pavement distresses. Unlike the PCR, roughness data for the PCI is collected using a mechanical roughness measuring device.

The PCI is a mathematical combination of the IRI and DMI (Distress Manifestation Index). The IRI is measured using the ARAN®. The DMI is calculated by combining the density and severity of all distresses using the following formula:

AC Pavement: $$DMI = 10*(208 - \sum_{k}^{N}(S_k + D_k) \times W_k) / 208$$

PCC Pavement: $$DMI = 10*(220 - \sum_{k}^{N}(S_k + D_k) \times W_k) / 220$$

Composite Pavement: $$DMI = 10*(196 - \sum_{k}^{N}(S_k + D_k) \times W_k) / 196$$

Surface Treated Pavement: $$DMI = 10*(135 - \sum_{k}^{N}(S_k + D_k) \times W_k) / 135$$

Where,

N is the number of distresses related to a given pavement type

$\mathbf{S}_k$ represents the severity rate of distress k

$\mathbf{D}_k$ represents the density rate of distress k

$\mathbf{W}_k$ is weighting factor of distress k

Note:

For **AC, COM,** and **PCC pavements**, the S_k and D_k used in the above formula are not directly from the surveyed severity and density rate for distress k. Instead, they are calculated based on the method described below:

Assume that SEV_k is the surveyed severity rate of distress k. If $SEV_k = 1$, then $S_k = 0.5$; if $SEV_k >= 2$, then $S_k = SEV_k - 1$

Assume that DEN_k is the surveyed density rate of distress k. If $DEN_k = 1$, then $D_k =$

0.5; if DEN_k >=2, then $D_k = DEN_k - 1$

The surveyed SEV_k and DEN_k are scaled from 1 to 5, therefore the range of S_k and D_k is 0.5–4,

For **Surface Treated pavements**, the S_k and D_k used in above formula are the same as surveyed SEV_k and DEN_k.

The surveyed SEV_k and DEN_k are scaled from 1 to 3; therefore the range of S_k and D_k is 1–3.

List of distress weighting factors for different pavement types are summarized in the following tables (Table 5.1.2 to 5.1.5).

Table 5.1.2 List of Distress Weighting Factor for AC

AC Pavement		
No.	Distress Type	*Weight (W_i)*
1	Ravelling and Coarse Aggregate Loss	3.0
2	Flushing	1.5
3	Rippling and Shoving	1.0
4	Wheel Track Rutting	3.0
5	Distortion	3.0
6	Longitudinal Wheel Track: Single and Multiple	1.5
7	Longitudinal Wheel Track: Alligator	3.0
8	Centreline: Single and Multiple Cracking	0.5
9	Centreline: Alligator Cracking	2.0
10	Pavement Edge: Single and Multiple Cracking	0.5
11	Pavement Edge: Alligator Cracking	1.5
12	Transverse: Half, Full and Multiple Cracking	1.0
13	Transverse: Alligator Cracking	3.0
14	Longitudinal Meandering and Midlane Cracking	1.0
15	Random Cracking	0.5

Table 5.1.3 List of Distress Weighting Factor for PCC

PCC Pavement		
No.	Distress Type	*Weight (W_i)*
1	Ravelling and Coarse Aggregate Loss	0.5
2	Polishing	1.5
3	Scaling	1.5
4	Potholing	1.0
5	Joint and Crack Spalling	2.0
6	Faulting	2.5
7	Distortion	1.0
8	Joint Sealant Loss	0.5
9	Transverse Joint Creep	0.5
10	Longitudinal Joint Separation	1.0
11	Joint Failure	3.0
12	Longitudinal and Meandering Cracking	2.0
13	Diagonal Corner and Edge Crescent	2.5
14	“D” Cracking	3.0
15	Transverse Cracking	2.0

Table 5.1.4 List of Distress Weighting Factor for Composite Pavement

COMPOSITE Pavement		
No.	Distress Type	*Weight (W_i)*
1	Ravelling and Coarse Aggregate Loss	3.0
2	Flushing	1.5
3	Spalling	2.0
4	Tenting/Cupping	2.5
5	Wheel Track Rutting	3.0
6	Distortion and Settlement	1.0
7	Joint Failures	3.0
8	Longitudinal Meandering (S&M) Cracking	2.0
9	Centreline: Single Cracking	0.5
10	Centreline: Multiple Cracking	1.5
11	Diagonal (S&M) Cracking	2.5
12	Transverse: Single Cracking	1.0
13	Transverse: Multiple Cracking	1.0
14	Map (S&M) Cracking	0.5
15	Transverse Joints: Sawed	0.5
16	Transverse Joints: Reflective	2.0

Table 5.1.5 List of Distress Weighting Factor for Surface Treated

ST(SURFACE TREATED) Pavement		
No.	Distress Type	*Weight (W_i)*
1	Cover Aggregate Loss	3.0
2	Streaking	1.0
3	Flushing	2.0
4	Potholing	1.0
5	Pavement Edge Break	2.0
6	Rippling and Shoving	2.0
7	Wheel Track Rutting	3.0
8	Distortion	3.0
9	Longitudinal Cracking	1.0
10	Transverse Cracking	0.5
11	Pavement Edge Cracking	1.0
12	Alligator Cracking	3.0

Correspondingly, the PCI is calculated using the following formula:

AC Pavement: $PCI = Max(0, Min(100, 13.75 + 9 \times DMI - 7.5 \times IRI))$

PCC Pavement: $PCI = Max(0, Min(100, 35.5 + 7 \times DMI - 11 \times IRI))$

Composite Pavement: $PCI = Max(0, Min(100, 20.5 + 8.5 \times DMI - 11 \times IRI))$

ST Pavement: $PCI = Max(0, Min(100, 12.75 + 9 \times DMI - 5.5 \times IRI))$

Where,

PCI is the pavement condition index

DMI is the subjective distress manifestation index, theoretically ranging from 0 to 10, where 0 indicates the worst condition well 10 represents the excellent condition

IRI is the international roughness index

PCI can be used for planning pavement rehabilitation. The following table is extracted from Reference [8]. This reference makes it clear that these are suggestions only and are subject to revision. Also, the reference states that there are other considerations like availability of funds and local conditions such as traffic volume, level of service, geometries, pavement structure, and pavement performance involved. These considerations, as well as the PCI, play an important role in the rehabilitation decision.

As the PCI decreased to a certain trigger value, the pavement requires treatments to bring up the PCI to a certain level to satisfy the needs of the public user. Table 5.1.6 and 5.1.7 below are the current MTO trigger values and the PCI Performance Targets for different functional classifications (subjected to change).

Table 5.1.6 MTO Trigger Level

	RCI	DMI	PCI
Freeways	6.0	7.3	65.0
Arterial	5.8	7.0	55.0
Collector	5.1	6.8	50.0
Local	5.1	6.8	45.0

Table 5.1.7 PCI Performance Targets

	Good		Fair		Poor	
	%	PCI	%	PCI	%	PCI
Freeway	70	75	30	74-66	0	65
Arterial	65	75	30	64-56	5	55
Collector	65	70	30	64-51	5	50
Local	60	65	30	59-46	10	45

5.2 PAVEMENT MANAGEMENT SYSTEMS

Pavement management system is a systematic methodology to assist in making decisions to find optimum strategies to provide, evaluate, and maintain the pavement network in an acceptable condition. Its objective is to facilitate the co-ordination of activities, provide consequences of decisions, and ensure consistency in decision-making. The activities involve the collection and storing of field data and the development and implementation of rehabilitation and maintenance programs [16], [17], [18].

5.2.1 COMPONENTS OF A PAVEMENT MANAGEMENT SYSTEM

The components of a pavement management system are shown in Figure 5.2.1, and it involves project level management and network level management. The diagram also illustrates that all activities from both project and network levels pavement management operate with a database [19].

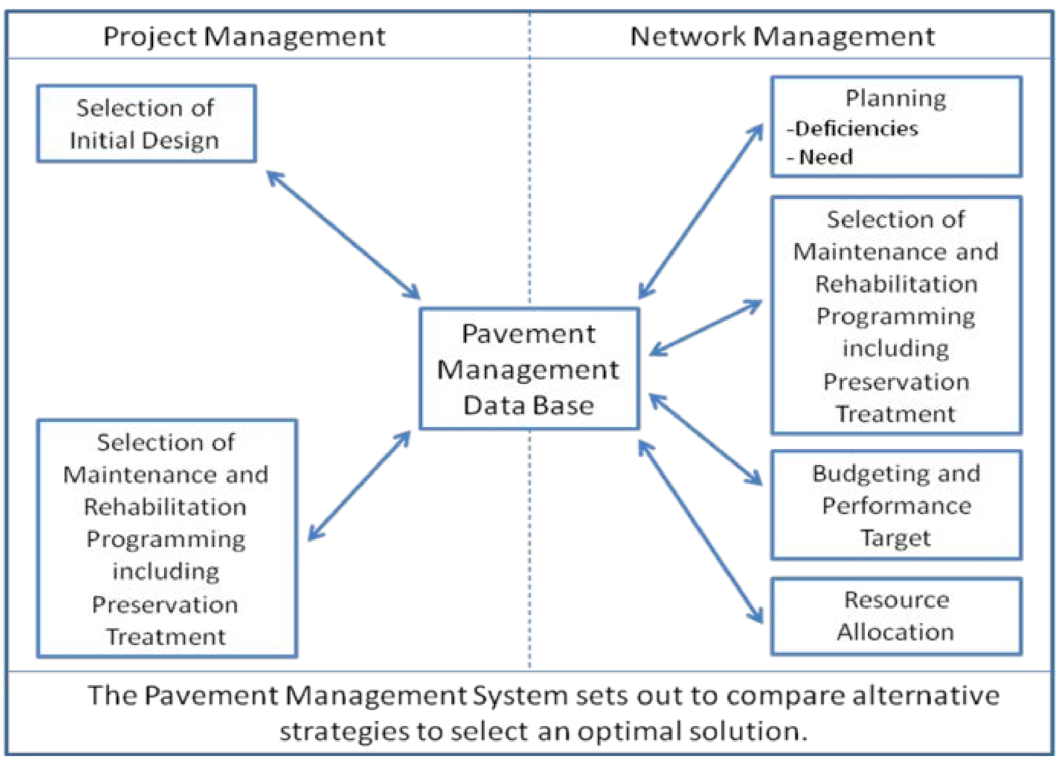


Figure 5.2.1 Structure of PMS

Database

Pavement evaluation data and highway inventory data is stored in a database. This data is used primarily in providing information to support the activities of pavement management, such as developing performance criteria. In addition, the data can be used to provide information which supports the research efforts of the agency. Research activities such as the development of new and improved pavement performance measures (e.g., PCI) and the development of performance prediction curves are impossible without a pavement management database.

Project Level Management

Project level pavement management is concerned with selecting the appropriate activity for an individual section of road. The process is similar to selecting the initial design and the appropriate rehabilitation or maintenance treatment for the road. In both cases, the process is to ensure the most cost-effective alternative being recommended.

Network Level Management

Network level management involves making decisions about funding the overall network. The main activities applied in network pavement management system include the twenty-five years pavement programming and investment planning and the five years published program. Planning and programming are complex procedures because they must consider the following:

- Funding for rehabilitation each year;
- Prediction of future pavement condition;
- Performance target and budget constraints;
- Construction quality / treatment effects;
- New construction technologies;
- Capacity of construction industry;
- Material availability, and
- Need of other assets such as bridges, etc.

The engineering evaluations of planning indicate the road needs repair. The budget realities of programming usually indicate there is not enough money to satisfy all needs in a given year. Trade-off is made between the optimal alternative selected in the project management component and the budgetary considerations in the network management component. Usually, priority lists are used to help make these trades-offs.

All network level pavement management systems use some kind of method in setting priorities. On one hand, the method can be as simple as ranking the sections in increasing order of pavement condition. On the other hand, the method can be as sophisticated as evaluating and selecting the optimum 10-year rehabilitation strategy for each section in the network based on the available funds. Both methods have been shown to work, and will be demonstrated in the next section. MTO uses concepts which employ both of these methods.

Subsequently, the network level pavement management optimization forms an integral part of the ministry’s overall asset management plan.

5.2.2 MTO PAVEMENT MANAGEMENT SYSTEMS

The first generation of Pavement Management System (PMS) used at MTO was developed in 1985, and the current PMS is the second generation referred to as PMS-2. MTO PMS-2 is an engineering tool used for managing the provincial roads network effectively and efficiently. It is part of an overall asset management framework for planning, investment analysis, and budgeting needs. PMS-2 assists decision-makers in finding optimum strategies to design, evaluate and maintain pavements in a cost-effective manner.

Salient Features of PMS-2

PMS-2 has the capability to:

- Collect pavement condition data by using objective data collection methods
- Evaluate pavement condition and predict its long-term performance

- Identify pavement repair needs, timing and alternative treatment strategies
- Analyze alternative investment scenarios and their impacts
- Determine an optimal maintenance and rehabilitation program
- Recommend a cost-effective priority list of projects.

Components of PMS-2

The major components of a comprehensive pavement management system include the following and are described in detail below:

- A network level database
- Pavement condition evaluation and performance measures
- Provincial performance targets and trigger values
- Performance prediction
- Treatment selection
- Prioritization and work programming
- Network-level analysis and budgeting
- Engineering feedback mechanism

Network Level Database

One of the key features of PMS-2 is the structure of its relational database. The database is comprised of detailed highway data tables developed for individual sections of roadway. The tables contain section-specific pavement definition information obtained from MTO's Integrated Highway Information System (IHIS) such as historical pavement condition data and as-built pavement structure details, and pavement condition data obtained from field pavement evaluation and data collection. As illustrated from Figure 5.2.2 below, the two-way interface permits the PMS-2 to receive pavement section information from IHIS and in turn, to provide IHIS with the needs, rehabilitation and optimization analysis results for each pavement section. [20]

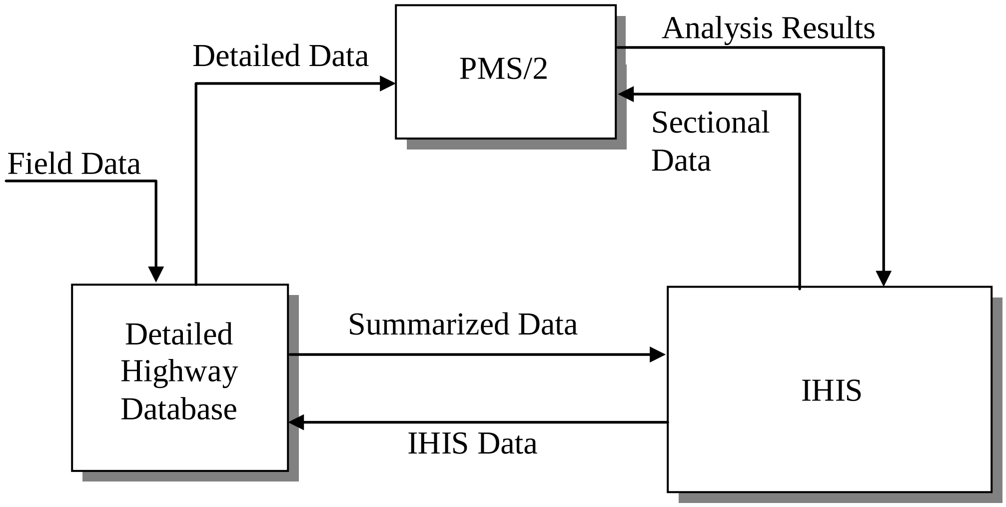


Figure 5.2.2 PMS-2 Integration with IHIS

Pavement Condition Evaluation and Performance Measures

Pavement condition evaluation in PMS-2 is made at the pavement section level. Current or most recent conditions, as well as the predicted future conditions in terms of pavement performance indices are summarized by the system.

Pavement condition surveys are integral to MTO's highway management program. The ministry recently implemented Pavement Distress Data Collection (PDDC) software to enhance the efficiency of manual pavement condition assessments throughout the province.

The new PDDC software, developed by MTO's I&IT group, is an application that runs on the user’s laptop during field inspections. The PDDC program is designed to integrate comprehensive data for various pavement distresses; which includes severity and density measures, historical maintenance records, and overall distress conditions calculations. This data is fed into PMS-2 to manage information about the provincial pavement network, plan rehabilitation projects, and coordinate investment decisions.

The new PDDC was fully implemented across the province in May 2005. The advantages of PDDC over the previous Pavement Condition Rating (PCR) program include: improved data accuracy, user-friendly electronic input, and a direct link to PMS-2 that enables data importing and exporting. The software also offers immediate data retrieval from PMS-2 (i.e. if a pavement designer wishes to download and refer to the previous evaluation of a pavement section), multiple navigation and data search options, standardized evaluation measures and distress ratings, and the ability to generate reports for a single pavement section or a group of sections. The implementation of this new PDDC software has noticeably enhanced the efficiency and reliability of current pavement distress investigations.

For future pavement conditions, they are developed by PMS-2 using prediction models.

Provincial Performance Targets and Trigger Values

The performance targets and trigger values are part of the evaluation of pavement performance, which was discussed in section 5.1.6 of this Manual. The provincial performance targets and trigger values are provided in Tables 5.1.6 and Table 5.1.7.

Performance Prediction

Pavement performance prediction models provide future RCI and DMI. Overall Pavement Condition Index (PCI) is calculated based on the predicted RCI and DMI. [20]

The mathematical model used is:

$$P = P_0 - e^{(a - b \times c^t)}$$

Where p = performance index, RCI or DMI

P_0 = P age 0

T= $\log_e$ (1/age)

a,b,c = model coefficients

Depending on the model coefficients, the shape of the performance curve can be different. Figure 5.2.3 below is an example of the default RCI and DMI prediction models.

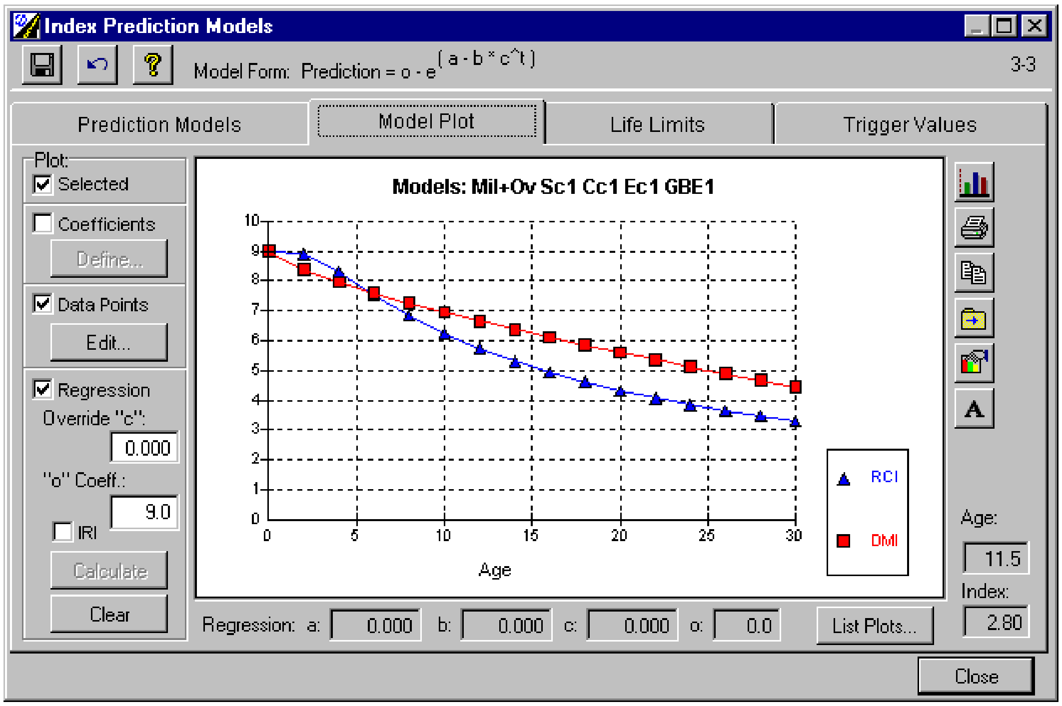


Figure 5.2.3 Example of RCI and DMI Prediction Models

<u>Treatment Selection</u>

Based on analysis of the current and future pavement conditions, a network level needs distribution is developed. Trigger or minimum acceptable index values are established and used to identify pavement sections performing below the trigger value. These sections are further analyzed for rehabilitation strategies. Figure 5.2.4 below is an example of decision tree in PMS-2.

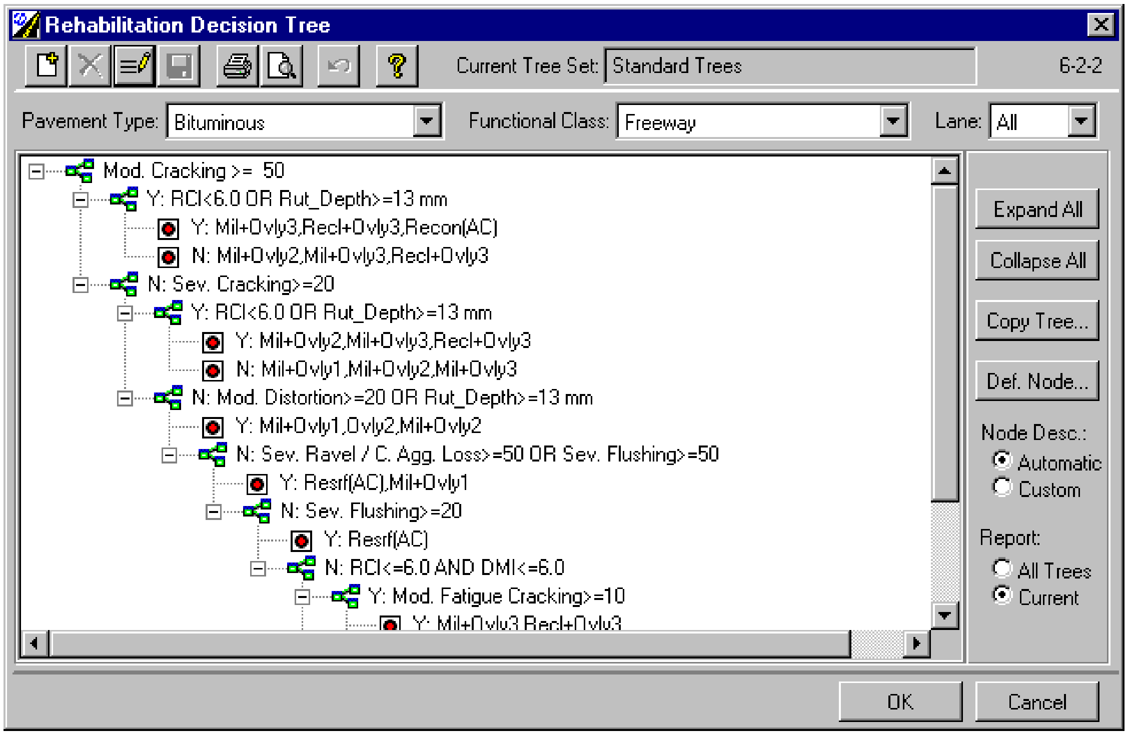


Figure 5.2.4 Example of Decision Tree for Treatment Selection in PMS-2

Prioritization and Work Programming

The network optimization analysis program in PMS-2 is used for budgeting, programming and planning analyses. The selection of treatment alternatives is performed using the marginal cost-effectiveness (MCE) method, in which the marginal cost-effectiveness of a treatment strategy is calculated as follow:

$$MCE_{is} = \frac{E_i - E_s}{C_i - C_s}$$

where:

MCE_{is} = marginal cost-effectiveness of strategy i given the selected strategy s,

E_i = effectiveness of strategy i,

C_i = cost of strategy i,

E_s = effectiveness of strategy s,

C_s = cost of strategy s.

The analysis initially selects treatments with maximum cost-effectiveness for each pavement section that is in needs. Alternative feasible treatments are then evaluated in order to maximize the MCE within the budget constraints.

Network-Level Analysis and Budgeting

The objective of network level analysis for planning is to estimate the required pavement preservation investments in the 1-10 year horizon. A second objective is to evaluate the impact of different budget streams on future network performance. These two types of analysis can be carried out using the effectiveness maximization method and the cost minimization method. More details can be found in Reference [20].

Engineering Feedback Mechanism

The Engineering Feedback Analysis is used to update the coefficients of the default performance index models and to analyze treatment effectiveness. These functions are designed as tools for the ongoing maintenance of the PMS-2.

5.2.3 OTHER PAVEMENT MANAGEMENT SYSTEMS

Many road agencies have developed their own pavement management systems to suit their own needs. Therefore, many types of pavement management systems exist. Each system has features that are unique. Data collection, budget decision-making, prioritization strategies, performance measures and trigger values are unique to each agency. But the purpose of pavement management system is the same, which is to maintain the database with wealth of pavement information for budgeting and prioritization of maintaining a healthy pavement network condition.

REFERENCES TO CHAPTER 5

[1] Phang, W.A., G.J. Chong; PAV-81-01: *Ontario Flexible Pavement Distress Assessment for Use in Pavement Management*, MTC R&D Branch, Downsview, 1982 (Reprint of paper published in Transportation Research Record 893).

[2] Chong, G.J., W.A. Phang, G.A. Wrong; SP-004: *Manual for Condition Rating of Flexible Pavements -Distress Manifestations*; MTC R&D Branch, Downsview, 1982.

[3] Chong, G.J., W.A. Phang, G.A. Wrong; SP-005: *Manual for Condition Rating of Rigid Pavements -Distress Manifestations*; MTC R&D Branch, Downsview, 1982.

[4] Chong, G.J., W.A. Phang, G.A. Wrong; SP-021: *Manual for Condition Rating of Surface-Treated Pavements -Distress Manifestations*; MTO R&D Branch, Downsview, 1988.

[5] Chong, G.J., W.A. Phang, G.A. Wrong; SP-022: *Flexible Pavement Condition Rating - Guidelines for Municipalities*; MTO R&D Branch, Downsview, 1988.

[6] Anderson, K.O.; *Pavement Surface Condition Rating Systems*; RTAC, Ottawa, 1987.

[7] Chong, G.J., G.A. Wrong; SP-025: *Manual for Condition Rating of Gravel Surface Roads*; MTO R&D Branch, Downsview, 1989.

[8] Hajek, J.J., W.A. Phang, G.A. Wrong, A. Prakash, G.M. Stott; PAV-86-02: *Pavement Condition Index (PCI) For Flexible Pavements*; MTC R&D Branch, Downsview, 1986.

[9] Paterson, W.D.O. 1987. *International Roughness Index: Relationship to Other Measures of Roughness and Riding Quality*, Transportation Research Record 1084, Transportation Research Board, Washington, D.C.

[10] E 274 - 85 - ASTM Standard, *Skid Resistance of Paved Surfaces Using a Full-Scale Tire*; Philadelphia.

[11] Schonfeld, R.; *Photo-Interpretation of Pavement Skid Resistance*; MTC R&D Branch, Downsview.

[12] Kazmierowski, T.J., "*Rutting of Ontario Expressways - Causes and Mitigation*" A.P.W.A. Conference, Toronto, 1987.

[13] Ministry Directive C- 145: *Asphalt Concrete Pavement Coring Procedures*; 86 04 01; Executive Director, Highway Engineering Division, Downsview.

[14] IR-42: Chong, G.J., M. Stott; *Evaluation of the Dynaflect and Pavement Design Procedures*; R&D Branch, Downsview, 1971.

[15] S. Chan and B. Lane; *Falling Weight Deflectometer (FWD) Testing Guideline; MERO Report # 19*, ISBN 0-7794-8720-6 (Print), MTO, Downsview, 2005.

[16] RTAC Pavement Management Committee; *Pavement Management Guide*, Ottawa, 1977.

[17] Pavement Management Study Working Group; *Outline of Concepts for Preparation of PMS Action Plans*; MTO, Downsview, 1986.

[18] Prakash, A. *Design and Development of a Pavement Management System for Large Network*; Proceedings of the International Conference on Roads and Road Transport Problems, India, 1988.

[19] Finn, F.N. Pavement Management Systems: *Selecting Maintenance Priorities*. Civil Engineering, ASCE, September 1983.

[20] Tom Kazmierowski, Zhiwei He and Brian Kerr. *A Second Generation PMS For the Ministry of Transportation of Ontario*. Fifth International Conference on Managing Pavements. Seattle, Washington, USA., August 2001.

(Blank Page)

APPENDIX A:

MTO PAVEMENT CONDITION EVALUATION FORMS

Ministry of Transportation Ontario

FLEXIBLE PAVEMENT CONDITION EVALUATION FORM

Location From: ______ **To:** ______

LHRS ☐☐☐☐☐ (BEGINS) ☐☐☐.☐ km (OFFSET) **Section Length** ☐☐.☐ km **District** ☐☐

Survey Date ☐☐ (YEAR) ☐☐ (MONTH) **PCR** ☐☐ **RCR** ☐.☐ **Traffic Direction** ☐ B:BOTH DIRECTIONS N:NORTH BOUND S:SOUTH BOUND E:EAST BOUND W:WEST BOUND **Facility** ☐ A: ALL LANES C:COLLECTOR E:EXPRESS O:OTHERS **Highway** ☐☐☐☐

Contract No. ☐☐☐☐ - ☐☐☐☐ **WP No.** ☐☐☐☐ - ☐☐ - ☐☐☐ **DMI** ☐ **Class** ☐ F:FREEWAY A:ARTERIAL C:COLLECTOR L:LOCAL

Ride Condition Rating (at 80 km/h)

10–8	**EXCELLENT** Smooth and pleasant
8–6	**GOOD** Comfortable
6–4	**FAIR** Uncomfortable
4–2	**POOR** Very rough and bumpy
2–0	**VERY POOR** Dangerous at 80km/h

Pavement			SEVERITY OF DISTRESS: Very Slight 1	Slight 2	Moderate 3	Severe 4	Very Severe 5	DENSITY OF DISTRESS (Extent of Occurrence %): Few <10 1	Intermittent 10-20 2	Frequent 20-50 3	Extensive 50-80 4	Throughout >80 5
SURFACE DEFECTS	Ravelling & C. Agg. Loss	1										
	Flushing	2										
SURFACE DEFORMATION	Rippling and Shoving	3										
	Wheel Track Rutting	4										
	Distortion	5										
CRACKING – Longitudinal Wheel Crack	Single and Multiple	6										
	Alligator	7										
CRACKING – Centre Line	Single and Multiple	8										
	Alligator	9										
CRACKING – Pavement Edge	Single and Multiple	10										
	Alligator	11										
CRACKING – Transverse	Half, Full and Multiple	12										
	Alligator	13										
CRACKING	Longtidinal Meander and Mildlane	14										
	Random	15										

Shoulders: DOMINANT TYPE	ONE ?	DISTRESS	LEFT SEVERITY Light 2	Med 3	Sever 4	LEFT DENSITY (Extent of Occurrence, %) <10 2	10-30 3	>30 4	RIGHT SEVERITY Light 2	Med 3	Sever 4	RIGHT DENSITY (Extent of Occurrence, %) <10 2	10-30 3	>30 4
PAVED FULL		Cracking												
		Pave Edge/ Curb Separation												
PAVED PARTIAL		Distortion												
SURFACE TREATED		Breakup/Separation												
		Edge Break												
PRIMED		Breakup												
GRAVEL														

Maintenance Treatment		EXTENT OF OCCURRENCE, %: <10 1	10-20 2	20-50 3	50-80 4	>80 5
PAVEMENT	Manual Pathcing					
	Machine Patching					
	Spray Patching					
	Rout and Seal Cracks					
	Chip Seal					
SHOULDERS	Manual Pathcing					
	Machine Patching					
	Rout and Seal Cracks					
	Chip Seal					

Distress Comments (items not coverred above) ______

Other Comments (e.g. Subsections, additional contracts items not coverred above) ______

Ministry of Transportation Ontario

RIGID PAVEMENT (Concrete Surface) CONDITION EVALUATION FORM

Location From: ______________________ **To:** ______________________

LHRS [ ] BEGINS [ ] OFFSET km **Section Length** [.] km **District** [ ]

Survey Date [ ] YEAR [ ] MONTH **PCR** [ ] **RCR** [.] **Traffic Direction** [ ] B:BOTH DIRECTIONS N:NORTH BOUND S:SOUTH BOUND E:EAST BOUND W:WEST BOUND **Facility** [ ] A: ALL LANES C:COLLECTOR E:EXPRESS O:OTHERS **Highway** [ ]

Contract No. [ ] - [ ] **WP No.** [ ] - [ ] - [ ] **DMI** [ ] **Class** [ ] F:FREEWAY A:ARTERIAL C:COLLECTOR L:LOCAL

Ride Condition Rating (at 80 km/h)

- 10
- 8 — **EXCELLENT** Smooth and pleasant
- 6 — **GOOD** Comfortable
- 4 — **FAIR** Uncomfortable
- 2 — **POOR** Very rough and bumpy
- 0 — **VERY POOR** Dangerous at 80km/h

Pavement			SEVERITY OF DISTRESS					DENSITY OF DISTRESS Extent of Occurrence %				
			Very Slight	Slight	Moderate	Severe	Very Severe	Few	Intermittent	Frequent	Extensive	Throughout
								<10	10-20	20-50	50-80	>80
			1	2	3	4	5	1	2	3	4	5
Surface Defects	Ravelling & C. Agg. Loss	1										
	Polishing	2										
	Scaling	3										
	Potholing	4										
	Joint and Crack Spalling	5										
Surface Deformation	Faulting (Stepping)	6										
	Distortion(sagging & slab warping)	7										
Joint Deficiencies	Joint Sealant Loss	8										
	Transverse Joint Creep	9										
	Longitudinal Joint Separation	10										
	Joint Failures (blow-ups)	11										
Cracking	Longitudinal & Meandering	12										
	Diagonal Corner & Edge Crescent	13										
	"D"	14										
	Transverse	15										

Distress Comments (items not coverred above) ______________________

Other Comments (e.g. Subsections, additional contracts items not coverred above) ______________________

Shoulders				LEFT						RIGHT					
Surface	WIDTH	ONE ?	DISTRESS	SEVERITY			DENSITY Occurrence, %			SEVERITY			DENSITY Occurrence, %		
				Light	Med	Sever	<10	10-30	>30	Light	Med	Sever	<10	10-30	>30
				2	3	4	2	3	4	2	3	4	2	3	4
Concrete	Fully		Cracking												
	Partially Paved		Pave Edge/ Curb Separation												
			Distortion												
Hot-Mix	Fully		Cracking												
	Partially Paved		Pave Edge/ Curb Separation												
			Distortion												
Surafce Treated	Fully		Cracking												
	Partially Paved		Pave Edge/ Curb Separation												
			Distortion												
Primed	Fully		Break Up												
	Partially														
Gravel															

Maintenance Treatment		EXTENT OF OCCURRENCE, %				
		<10	10-20	20-50	50-80	>80
		1	2	3	4	5
PAVEMENT	Manual Pathcing, Hot or Cold Mix					
	Machine Patching, Partial/Full width					
	Microsurfacing					
	Grooving					
	Rout & Seal Cracks or Joints					
	Concrete Joint/Slab Replacement					
SHOULDERS	Manual Pathcing, Hot or Cold Mix					
	Machine Patching, Partial/Full width					
	Spray Pathing / Chip Seal					
	Seal Cracks or Joints					

Evaluated by: ______________________

Ministry of Transportation Ontario

SURFACE TREATED PAVEMENT CONDITION EVALUATION FORM

Location From: ____________ **To:** ____________

LHRS [ ][ ][ ][ ][ ] BEGINS [ ][ ][.][ ] OFFSET km **Section Length** [ ][ ][.][ ] km **District** [ ][ ]

Survey Date [ ][ ] YEAR [ ][ ] MONTH **PCR** [ ][ ] **RCR** [ ][.][ ] **Traffic Direction** [ ] B:BOTH DIRECTIONS N:NORTH BOUND S:SOUTH BOUND E:EAST BOUND W:WEST BOUND **Facility** [ ] A: ALL LANES C:COLLECTOR E:EXPRESS O:OTHERS **Highway** [ ][ ][ ][ ]

Contract No. [ ][ ][ ][ ] - [ ][ ][ ][ ] **WP No.** [ ][ ][ ][ ] - [ ][ ] - [ ][ ][ ] **DMI** [ ] **Class** [ ] F:FREEWAY A:ARTERIAL C:COLLECTOR L:LOCAL

Ride Condition Rating (at 80 km/h)

10	
8	**EXCELLENT** Smooth and pleasant
6	**GOOD** Comfortable
4	**FAIR** Uncomfortable
2	**POOR** Very rough and bumpy
0	**VERY POOR** Dangerous at 80km/h

Pavement			SEVERITY OF DISTRESS			DENSITY OF DISTRESS Extent of Occurrence %		
			Slight	Moderate	Severe	Intermittent	Frequent	Extensive
						<20	20-50	>50
			1	2	3	1	2	3
Surface Defects	Ravelling & C. Agg. Loss	1						
	Streaking	2						
	Flushing	3						
	Potholes	4						
	Pavement Edge Break	5						
Surface Deformation	Rippling	6						
	Wheel Track Rutting	7						
	Distortion	8						
Cracking	Longitudinal	9						
	Transverse	10						
	Pavement Edge	11						
	Alligator	12						

Shoulders	LEFT						RIGHT					
	SEVERITY			DENSITY Extent of Occurrence, %			SEVERITY			DENSITY Extent of Occurrence, %		
	Light	Med	Sever	<10	10-30	>30	Light	Med	Sever	<10	10-30	>30
	2	3	4	2	3	4	2	3	4	2	3	4
Encroaching Grow th												
Poor Cross-fall												

Maintenance Treatment For Pavement	EXTENT OF OCCURRENCE, %		
	<20	20-50	>50
	1	2	3
Manual Pathcing			
Machine Patching			
In Site Recycling			
Manual Spray Patching			
Manual Chip Seal			
Machine Chip Seal			
Fog Seal			
Manual Burn & Seal			

Distress Comments (items not coverred above) ____________

Other Comments (e.g. Subsections, additional contracts items not coverred above) ____________

Evaluated by: ____________

Ministry of Transportation Ontario

COMPOSITE PAVEMENT CONDITION EVALUATION FORM

Location From: ____________________ **To:** ____________________

LHRS [ ] BEGINS [.] OFFSET km **Section Length** [.] km **District** [ ]

Survey Date [ ] YEAR [ ] MONTH **PCR** [ ] **RCR** [.] **Traffic Direction** [ ] B:BOTH DIRECTIONS N:NORTH BOUND S:SOUTH BOUND E:EAST BOUND W:WEST BOUND **Facility** [ ] A: ALL LANES C:COLLECTOR E:EXPRESS O:OTHERS **Highway** [ ]

Contract No. [ ] - [ ] **WP No.** [ ] - [ ] - [ ] **DMI** [ ] **Class** [ ] F:FREEWAY A:ARTERIAL C:COLLECTOR L:LOCAL

Ride Condition Rating (at 80 km/h)

- 10
- **EXCELLENT** Smooth and pleasant
- 8
- **GOOD** Comfortable
- 6
- **FAIR** Uncomfortable
- 4
- **POOR** Very rough and bumpy
- 2
- **VERY POOR** Dangerous at 80km/h
- 0

Pavement			SEVERITY OF DISTRESS: Very Slight	Slight	Moderate	Severe	Very Severe	DENSITY OF DISTRESS (Extent of Occurrence %): Few	Intermittent	Frequent	Extensive	Throughout
								<10	10-20	20-50	50-80	>80
			1	2	3	4	5	1	2	3	4	5
Surface Defects	Ravelling & C. Agg. Loss	1										
	Flushing	2										
	Spalling	3										
Surface Deformation	Tenting/Cupping	4										
	Wheel Track Rutting	5										
	Distortion & settlement	6										
	Joint Failures	7										
Cracking	Longitudinal, Meandering - Single & Multiple	8										
	Centerline - single	9										
	Centerline - Multiple	10										
	Diag./Corner/Edge Cres. - Single & Multiple	11										
	Transverse - Single	12										
	Transverse - multiple	13										
	Map - Single & Multiple	14										
Transverse Joints	Saw ed	15										
	Reflective	16										

Distress Comments (items not coverred above) ____________________

Other Comments (e.g. Subsections, additional contracts items not coverred above) ____________________

Shoulders				LEFT SEVERITY			LEFT DENSITY Occurrence, %			RIGHT SEVERITY			RIGHT DENSITY Occurrence, %		
Surface	WIDTH	ONE ?	DISTRESS	Light	Med	Sever	<10	10-30	>30	Light	Med	Sever	<10	10-30	>30
				2	3	4	2	3	4	2	3	4	2	3	4
Concrete	Fully		Cracking												
	Partially Paved		Pave Edge/ Curb Separation												
			Distortion												
Hot-Mix	Fully		Cracking												
	Partially Paved		Pave Edge/ Curb Separation												
			Distortion												
Surafce Treated	Fully		Cracking												
	Partially Paved		Edge Break/ Separation												
			Break-up												
Primed	Fully		Break Up												
	Partially														
Gravel															

Maintenance Treatment		EXTENT OF OCCURRENCE, % <10	10-20	20-50	50-80	>80
		1	2	3	4	5
PAVEMENT	Manual Pathcing, Hot or Cold Mix					
	Machine Patching, Partial/Full width					
	Microsurfacing					
	Grooving					
	Rout & Seal Cracks or Joints					
	Concrete Joint/Slab Replacement					
SHOULDERS	Manual Pathcing, Hot or Cold Mix					
	Machine Patching, Partial/Full width					
	Spray Pathing / Chip Seal					
	Seal Cracks or Joints					

Evaluated by: ____________________

APPENDIX B:

MTO GUIDE FOR DESCRIBING THE SEVERITY OF DISTRESSES

(Blank Page)

ASPHALTIC CONCRETE PAVEMENTS - Guide for Describing Severity of Distresses

<table>
<tr><th colspan="4" rowspan="2">Distress Type</th><th colspan="5">Severity of Distress</th></tr>
<tr><th>1
Very Slight</th><th>2
Slight</th><th>3
Moderate</th><th>4
Severe</th><th>5
Very Severe</th></tr>
<tr><td colspan="2" rowspan="2">Surface Defects</td><td>1</td><td>Ravelling and Coarse Agg. Loss and Potholes</td><td>Barely noticeable loss.</td><td>Noticeable loss.</td><td>Pockmarks well-spaced, shallow disintegration, open- textured look.</td><td>Pockmarks closely-spaced, disintegration with small potholes.</td><td>Disintegration with large potholes.</td></tr>
<tr><td>2</td><td>Flushing</td><td>Very faint colouring (veining).</td><td>Colouring visible (interconnected veining).</td><td>Distinctive appearance with free asphalt.</td><td>Free asphalt on surface; has wet look.</td><td>Wet look with tire noise like driving on wet pavement surface.</td></tr>
<tr><td colspan="2" rowspan="3">Surface Deformations</td><td>3</td><td>Rippling and Shoving</td><td>Barely noticeable. Washboard effect.</td><td>Noticeable. Washboard effect.</td><td>Bumpy. Washboard effect.</td><td>Very bumpy. Pronounced washboard effect.</td><td>Large humps may cause poor control of car.</td></tr>
<tr><td>4</td><td>Wheel Track Rutting</td><td>3-6 mm

Usually no longitudinal cracks.</td><td>7-12 mm

May include single longitudinal cracks.</td><td>13-19 mm

May include starting multiple longitudinal cracks.

Dual rutting may begin to be visible.</td><td>20-25 mm

May include multiple longitudinal cracks.

May include dual rutting.</td><td>> 25 mm

May include multiple longitudinal cracks.

Usually with dual rutting.</td></tr>
<tr><td>5</td><td>Distortion</td><td>Barely noticeable swaying motion.</td><td>Fairly noticeable bump or drop.
Good control of car.</td><td>Noticeable bump or drop.

Good/fair control of car.</td><td>Significant bounce.

Fair control of car.</td><td>Excessive and harsh bounce.

Poor control of car.</td></tr>
<tr><td rowspan="10">Cracking</td><td rowspan="6">Single and Multiple</td><td>6</td><td>Longitudinal Wheel Track</td><td rowspan="5">Width <3 mm.

Single hairline cracks.

Transverse cracks may be partial or full width.</td><td rowspan="5">3-12 mm width.

Single cracks.

Transverse cracks may be partial or full width.</td><td rowspan="5">13-19 mm width for single cracks
or
multiple cracks starting.

Transverse cracks may also have slight cupping or lipping. (Barely noticeable bump.)</td><td rowspan="5">20-25 mm width for single cracks
or
multiple cracks; spalling begins to develop.

Transverse cracks may also have moderate cupping or lipping. (Noticeable bump.)</td><td rowspan="5">> 25 mm wide for single cracks
or
multiple cracks with spalling developed. May begin to alligator.

Transverse cracks may also have severe cupping or lipping. (Bump or thump.)</td></tr>
<tr><td>8</td><td>Centreline</td></tr>
<tr><td>12</td><td>Transverse (half, full & multiple)</td></tr>
<tr><td>14</td><td>Meander & Midlane</td></tr>
<tr><td>15</td><td>Random and/or Map</td></tr>
<tr><td>10</td><td>Pavement Edge</td><td>Width <3 mm.

Single crack or single wave formation.</td><td>3-12 mm width.

Single crack or two parallel cracks up to 0.3 m from edge.</td><td>Multiple cracks extend up to 0.6 m from edge.</td><td>Multiple cracks extend up to 1.5 m from edge.</td><td>Multiple cracks extend over 1.5 m from edge.

Alligator pattern forming.</td></tr>
<tr><td rowspan="4">Alligator</td><td>7</td><td>Longitudinal Wheel Track</td><td rowspan="4">Alligator pattern forming.

May include depression up to 6 mm.</td><td rowspan="4">Alligator pattern established with block corners fracturing.

May include depression 7-12 mm.</td><td rowspan="4">Alligator pattern established with spalling of blocks.

May include depression 13-19 mm.</td><td rowspan="4">Blocks begin to lift. Small potholes from missing blocks.

May include depression 20-25 mm.</td><td rowspan="4">Polygon blocks lifting. Potholes from missing blocks.

May include depression >25 mm.</td></tr>
<tr><td>9</td><td>Centreline</td></tr>
<tr><td>11</td><td>Pavement Edge</td></tr>
<tr><td>13</td><td>Transverse</td></tr>
</table>

Note: Crack width should be determined during the period from May to October.
Do not report routed and sealed cracks; these will be reported as Maintenance Treatment.

ASPHALTIC CONCRETE PAVEMENTS

Pavement Distresses

Guide for Describing Extent of Pavement Distresses

Class		All Distresses Except Transverse Cracking*	Transverse Cracking Only
1	Few	<10%	Cracks (full and/or half cracks) are more than about 40 m apart.
2	Intermittent	10-20%	No set pattern. Cracks (full and/or half) are about 30 to 40 m apart.
3	Frequent	20-50%	A set pattern. Cracks (full and/or half) are about 20 to 30 m apart.
4	Extensive	50-80%	Rather regular pattern. Cracks (full and/or half are about 10 to 20 m apart).
5	Throughout	80-100%	Regular pattern. Cracks (full and/or half) are less than about 10 m apart.

** Based on percent of surface area within the PMS section affected by distress*

Shoulder Distresses

Guide for Describing Severity of Shoulder Distresses

Type of Distress	Severity of Distress	
	1 Moderate	2 Severe
Cracking	Multiple slight to moderate cracks developed.	Multiple moderate to severe cracks developed.
Pavement Edge - Shoulder (Curb) Separation	Single cracks. Width: 13-19 mm.	Single crack width over 20 mm or multiple cracks.
Distortion	Noticeable edge curling, depression or heaving. No major cracks.	Obvious edge curling, depression or heaving. Multiple cracks.
Breakup	Disintegration with small potholes up to 150 mm.	Disintegration with potholes >150 mm.
Edge Break	Edge cracks with some loss of material.	Edge breaking with extensive loss of material.

Notes
Based on "Manual for Condition Rating of Flexible Pavements - Distress Manifestations", SP-024, Ministry of Transportation of Ontario, 1989.
In case of any discrepancies with the above Manual, this guide should govern.

Ministry of Transportation of Ontario
Pavements and Foundations Section
Pavement Management Unit
June 1995

CONCRETE PAVEMENTS - Guide for Describing Severity of Distresses

Distress Type		Severity of Distress: 1 Very Slight	2 Slight	3 Moderate	4 Severe	5 Very Severe
Surface Defects	1 Ravelling and Coarse Aggregate Loss	Barely noticeable loss.	Noticeable loss.	Pockmarks well-spaced, shallow disintegration, open-textured look.	Pockmarks closely-spaced, disintegration with small potholes.	Disintegration with large potholes.
Surface Defects	2 Polishing	Barely noticeable.	Noticeable dull finish.	Distinctive dull finish.	Glossy mirror finish.	Surface has highly polished appearance.
Surface Defects	3 Scaling	Barely noticeable.	Noticeable.	Open-textured look but very shallow.	Disintegration in closely-spaced, shallow patches.	Disintegration in large, shallow patches.
Surface Defects	4 Potholing	Barely noticeable. Resembles coarse aggregate loss.	Disintegration of surrounding materials.	Pothole 25 to 75 mm wide and deep. May affect ride.	Pothole 76-150 mm wide and deep. Begins to affect ride.	Pothole >150 mm wide and deep. Affects ride.
Surface Defects	5 Joint and Crack Spalling	Small fractures at joints or cracks.	Small cracks at joints or cracks, noticeable loss of materials.	Loss of materials; hole more than 75 mm wide, 75 mm deep. May affect ride.	Fractured edges, much loss of materials and potholing. Begins to affect ride.	Fractured edges, great loss of materials and potholing. Affects ride.
Surface Deformations	6 Faulting (Stepping)	Barely noticeable (<2 mm).	2-4 mm	5-8 mm	9-13 mm	>13 mm
Surface Deformations	7 Distortion (Sagging and Slab Warping)	Barely noticeable swaying motion.	Fairly noticeable bump or drop.	Noticeable bump or drop. Good control of car.	Significant bounce. Fair control of car.	Excessive and hard bounce. Poor control of car.
Joint Deficiencies	8 Joint Sealant Loss	Barely popped out or breaking.	Sealant broken and beginning to pull out (up to 30 cm).	Sealant broken and pulled out by up to 50% of its length.	Sealant broken and pulled out by up to 80% of its length.	Sealant completely broken/pulled out more than 80% of its length. Ineffective as a sealant.
Joint Deficiencies	9 Transverse Joint Creep	Joints barely out of line.	Joints noticeably out of line (<19 mm).	Joints 19-25 mm out of line.	Joints 26-50 mm out of line.	Joints >50 mm out of line.
Joint Deficiencies	10 Longitudinal Joint Separation	Not applicable.	Not applicable.	Up to 25 mm	26-50 mm	>50 mm
Joint Deficiencies	11 Joint Failures (Blow-Ups)	Not applicable.	Not applicable.	Not applicable.	Pavement fractures into blocks requiring immediate repairs. Patching near the joint. Moderate distortion.	Pavement fractures into large blocks requiring immediate repairs. Pavement extends up to 3 m from the joint. Severe distortion.
Cracking	12 Longitudinal & Meandering	Width <3 mm. *Transverse cracks may be partial or full width.*	3-12 mm wide.	13-19 mm wide. With or without spalling and faulting.	20-25 mm wide. With spalling and faulting.	>25 mm wide. With spalling and faulting.
Cracking	13 Diagonal Corner & Edge Crescent					
Cracking	15 Transverse					
Cracking	14 "D"	<3 mm wide single crack.	3-12 mm wide single crack.	13-19 mm wide single crack or multiple cracks <13 mm wide.	19-25 mm wide single crack or multiple cracks 13-19 mm wide.	>25 mm wide single crack or multiple cracks 20-25 mm wide.

Note: Crack width should be determined during the period from May to October.

CONCRETE PAVEMENTS

Pavement Distresses

Guide for Describing Extent of Pavement Distresses

Class	All Distresses Except Transverse Cracking*	Transverse Cracking Only
1 Few	<10%	Cracks are more than about 80 m apart.
2 Intermittent	10-20%	No set pattern. Cracks are about 60 to 80 m apart.
3 Frequent	20-50%	A set pattern. Cracks are about 40 to 60 m apart.
4 Extensive	50-80%	Rather regular pattern. Cracks are about 20 to 40 m apart.
5 Throughout	80-100%	Regular pattern. Cracks are less than about 20 m apart.

** Based on percent of surface area within the PMS section affected by distress*

Shoulder Distresses

Guide for Describing Severity of Shoulder Distresses

Type of Distress	Severity of Distress	
	1 Moderate	2 Severe
Cracking	Multiple slight to moderate cracks developed.	Multiple moderate to severe cracks developed.
Pavement Edge - Shoulder (Curb) Separation	Single cracks. Width: 13-19 mm.	Single crack. Width over 20 mm or multiple cracks.
Distortion	Noticeable edge curling, depression or heaving. No major cracks.	Obvious edge curling, depression or heaving with multiple cracks.
Breakup	Disintegration with small potholes up to 150 mm.	Disintegration with potholes >150 mm.
Edge Break	Edge cracks with some loss of material.	Edge breaking with extensive loss of material.

Notes
Based on "Manual for Condition Rating of Rigid Pavements - Concrete Surface and Composite Distress Manifestations", SP-005 (revised), Ministry of Transportation of Ontario, 1995. In case of any discrepancies with the above Manual, this guide should govern.

Ministry of Transportation of Ontario
Pavements and Foundations Section
Pavement Management Unit
June 1995

COMPOSITE PAVEMENTS - Guide for Describing Severity of Distresses

	Distress Type	Severity of Distress 1 Very Slight	 2 Slight	 3 Moderate	 4 Severe	 5 Very Severe
Surface Defects	1 Ravelling and Coarse Aggregate Loss and Potholes	Barely noticeable loss.	Noticeable loss.	Pockmarks well spaced, shallow disintegration, open-textured look.	Pockmarks closely-spaced, disintegration with small potholes.	Disintegration with large potholes.
	2 Flushing	Very faint colouring (veining).	Colouring visible. (interconnected veining).	Distinctive appearance with free asphalt.	Free asphalt on surface, has wet look.	Wet look with tire noise like driving on wet pavement surface.
	3 Joint and Crack Spalling	Small fractures at joints or cracks.	Small cracks at joints or cracks, noticeable loss of materials.	Loss of materials. Potholes up to 75 mm wide and deep. May affect ride.	Fractured edges, much loss of materials and potholing. Potholes 76-150 mm wide and deep. Begins to affect ride.	Fractured edges, great loss of materials and potholing. Potholes >150 mm wide and deep. Affects ride.
Surface Deformations	4 Tenting/Cupping (Stepping)	<3 mm displacement. Barely noticeable.	3-6 mm. Noticeable bump.	7-12 mm. Noticeable bump.	13-19 mm. Bump or thump.	>19 mm. Bump or thump.
	5 Wheel Track Rutting	3-6 mm Barely noticeable.	7-12 mm. Somewhat noticeable.	13-19 mm. Dual rutting may begin to be visible.	20-25 mm. Usually with dual rutting.	>25 mm. Usually with dual rutting.
	6 Distortion and Settlement	Barely noticeable swaying motion.	Fairly noticeable bump or drop.	Noticeable bump or drop. Good control of car.	Significant bounce. Fair control of car.	Excessive and harsh bounce. Poor control of car.
	7 Joint Failures	Not Applicable.	Not Applicable.	Cupping/tenting, multiple cracks radiating from joint.	Multiple cracks and spalling. Some vehicle bounce.	Bump or dip at joint break-up. Significant bounce. Temporary patching may have been done.
Cracking	8 Longitudinal (Single & Mult.) 11 Diag., Corner & Edge Cres. (Sing. & Mult.)	Width <3 mm. Single cracks.	3-12 mm wide. Single cracks.	13-19 mm wide for single cracks or multiple cracks starting.	20-25 mm wide for single cracks or multiple cracks with initial signs of spalling.	>25 mm wide for single cracks or multiple cracks with spalling.
	9 Centreline Single 12 Transverse Single	Width <3 mm.	3-12 mm wide.	13-19 mm wide. *For transverse cracks, tenting and cupping developing.*	20-25 mm wide. *For transverse cracks, tenting and cupping developed.*	>25 mm wide. *For transverse cracks, tenting/cupping/spalling well developed.*
	10 Centreline Multiple 13 Transverse Multiple	Not Applicable.	Not Applicable.	3-12 mm wide. Secondary cracks may radiate from single crack. *For transverse cracks, tenting/ cupping may start.*	13-19 mm wide. Secondary cracks may radiate from single cracks. *For transverse cracks, tenting/ cupping developed.*	>19 mm wide. *For transverse cracks, tenting/ cupping/spalling well developed.*
	14 Map (Single & Multiple)	Width <3 mm. Single short cracks.	Width 3-12 mm. Interconnected single cracks.	Single crack width 13-19 mm. Multiple cracks begin to develop.	Single crack width 20-25 mm. Multiple interconnected cracks with some spalling.	Single cracks width >25 mm. Multiple interconnected cracks with spalling.
Transverse Joints	15 Transverse Joints (Sawed)	Some sealant bond failure. Single secondary cracks developing <3 mm in width.	Some loss of sealant or bond failure. Single secondary cracks 3-12 mm.	Minor loss of sealant or bond failure. Single secondary cracks 3-12 mm. Multiple cracks start to develop.	Major loss of sealant. Single secondary cracks 13-19 mm or multiple cracks.	Nearly complete loss of sealant. Single secondary cracks >19 mm or multiple cracks.
	15 Transverse Joints (Reflective)	Joint opening <3 mm. Single cracks.	Joint opening 3-12 mm. Single cracks.	Joint opening 13-19 mm or secondary cracks or tenting/ cupping/spalling starting.	Joint opening 20-25 mm or secondary cracks 14-19 mm or tenting/cupping/spalling developed.	Joint opening > 25 mm or secondary cracks >19 mm wide. Tenting/cupping/spalling and/or secondary cracks well developed.

Note: Crack width should be determined during the period from May to October.

COMPOSITE PAVEMENTS

Pavement Distresses

Guide for Describing Extent of Pavement Distresses

Class	All Distresses Except Transverse Cracking*	Transverse Cracking Only
1 **Few**	<10%	Cracks are more than about 80 m apart.
2 **Intermittent**	10-20%	No set pattern. Cracks are about 60 to 80 m apart.
3 **Frequent**	20-50%	A set pattern. Cracks are about 40 to 60 m apart.
4 **Extensive**	50-80%	Rather regular pattern. Cracks are about 20 to 40 m apart).
5 **Throughout**	80-100%	Regular pattern. Cracks are less than about 20 m apart.

** Based on percent of surface area within the PMS section affected by distress*

Shoulder Distresses

Guide for Describing Severity of Shoulder Distresses

Type of Distress	Severity of Distress	
	1 Moderate	2 Severe
Cracking	Multiple slight to moderate cracks developed.	Multiple moderate to severe cracks developed.
Pavement Edge - Shoulder (Curb) Separation	Single cracks. Width: 13-19 mm.	Single cracks. Width over 20 mm or multiple cracks.
Distortion	Noticeable edge curling, depression or heaving. No major cracks.	Obvious edge curling, depression or heaving with multiple cracks.
Breakup	Disintegration with small potholes up to 150 mm.	Disintegration with potholes >150 mm.
Edge Break	Edge cracks with some loss of material.	Edge breaking with extensive loss of material.

Notes
Based on "Manual for Condition Rating of Rigid Pavements - Concrete Surface and Composite Distress Manifestations", SP-005 (revised), Ministry of Transportation of Ontario, 1995. In case of any discrepancies with the above Manual, this guide should govern.

Ministry of Transportation of Ontario
Pavements and Foundations Section
Pavement Management Unit
June 1995

SURFACE-TREATED PAVEMENTS - Guide for Describing Severity of Distresses

Distress Type			Severity of Distress: 1 Slight	2 Moderate	3 Severe
Surface Defects	1	Loss of Cover Aggregate	Noticeable loss of cover aggregate.	Large patches of aggregate loss, leaving black spots.	Complete loss of cover aggregate.
	2	Streaking	Single or double lines streaking on the pavement surface.	Multiple lines streaking on the pavement surface.	Alternate lean and heavy lines streaking on the pavement surface.
	3	Flushing	Colouring visible (interconnected veining).	Distinctive appearance with free asphalt.	Free asphalt on surface has wet look.
	4	Potholes	Small and shallow (<100 mm in diameter).	100 to 300 mm wide; usually deeper than 50 mm.	> 300 mm wide; usually deeper than 100 mm.
	5	Pavement Edge Break	Small breaks (<250 mm). Few or no missing pieces.	Breaks between 250 and 750 mm. Pieces missing.	Large breaks (>750 mm). Large pieces missing.
Surface Deformation	6	Rippling	Noticeable washboard effect.	Bumpy with washboard effect, ridges and valleys.	Very bumpy with pronounced washboard. effect.
	7	Wheel Track Rutting	7-12 mm May include single longitudinal cracks.	13-19 mm May include starting multiple longitudinal cracks.	>19 mm May include multiple longitudinal cracks.
	8	Distortion	Fairly noticeable bump or drop.	Noticeable bump or drop. Good control of car.	Significant bounce. Fair control of car.
Cracking	9	Longitudinal Single & Multiple	Width up to 12 mm. Single cracks.	13-19 mm width for single cracks or multiple cracks starting. *Transverse cracks may also have slight cupping or lipping. (Barely noticeable bump.)*	>19 mm wide for single cracks or multiple cracks; spalling begins to develop. *Transverse cracks may also have moderate cupping or lipping. (Noticeable bump.)*
	10	Transverse Single & Multiple			
	11	Pavement Edge Single & Multiple	<300 mm from pavement edge. Single or two parallel cracks.	Multiple cracks extending between 300 to 900 mm from edge. Multiple cracks (with connecting cracks).	Multiple cracks extending over 900 mm from pavement edge. Alligatoring pattern forming.
	12	Alligator	Alligator pattern established with block corners fracturing. May include depression up to 12 mm.	Alligator pattern established with spalling of polygon blocks. May include depression 13-19 mm.	Blocks begin to lift, small potholes from missing blocks. May include depression >19 mm.

Note: *Crack width should be determined during the period from May to October.*

SURFACE-TREATED PAVEMENTS

Pavement Distresses

Guide for Describing Extent of Pavement Distresses

Class		All Distresses Except Transverse Cracking*	Transverse Cracking Only
1	Intermittent	<20%	No set pattern. Cracks (full and/or half) are about 30 to 40 m apart.
2	Frequent	20-50%	A set pattern. Cracks (full and/or half) are about 20 to 30 m apart.
3	Extensive	>50%	Rather regular pattern. Cracks (full and/or half are about 10 to 20 m apart).

** Based on percent of surface area within the PMS section affected by distress*

Shoulder Distresses

Guide for Describing Shoulder Distresses

Type of Distress	Description of Distress
Encroaching Growth	Vegetation growth overruns shoulder and reaches or approaches close to pavement edge.
Poor Cross-Fall	Shoulders higher than pavement resulting in trapped surface water on the pavement edge or on shoulder immediately adjacent to it.

Notes
Based on "Manual for Condition Rating of Surface-Treated Pavements - Distress Manifestations", SP-021, Ministry of Transportation of Ontario, 1989.
In case of any discrepancies with the above Manual, this guide should govern.

Ministry of Transportation of Ontario
Pavements and Foundations Section
Pavement Management Unit
June 1995

(Blank Page)

ISBN: 978-1-4435-2872-6 (Print)
ISBN: 978-1-4435-2873-3 (PDF)

Ontario

Ministry of Transportation

MTO MERO

Materials Engineering and Research Office