AN EVALUATION OF SELECTED ASPHALT PAVEMENTS IN THE CITY OF MONTREAL

by

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Abstract

Pavements are generally analyzed as elastic layered structures, and designed empirically. Currently, more than 25% of road pavements in Canada fail prematurely (i.e. shortly after they are constructed) whereas only 60% reach their actual design life (C-SHRP, 2002). Pavement failures are generally displayed at the top of the asphalt layer. These deteriorations rarely initiate at the surface as the different layers interact co-dependently in resisting the applied loads. The failure of any one of them will weaken the pavement and reflect at the surface as deteriorations.

The City of Montreal is experiencing extensive premature pavement failures mostly in the form of potholes and fatigue cracks which have been treated and repaired using different surface restoration techniques. Unfortunately, these solutions do not address the real problem and are only temporary.

This research reviews the pavement repair and maintenance techniques used in the City of Montreal, explores their adequacy, and demonstrates how the current design procedures have yielded weak pavements that are short on thickness and proper drainage. The objective of this thesis is also to determine the main reason why Montreal road pavements fail so prematurely in terms of the current design, repair and maintenance practices. Furthermore, this research project includes a case study of the road pavement condition in the City of Montreal, coupled with a comparative approach to the standards of practice used in both of Quebec and Ontario provinces.

Résumé

Les chaussées sont généralement considérées comme des structures superposées. Elles sont analysées selon la loi de Hooke et conçues de façon empirique. Actuellement, plus de 25% des routes au Canada présentent des dommages prématurées alors que 60% atteignent leur espérance de vie (C-SHRP, 2002). Bien que les détériorations se manifestent à la surface de la chaussée, elles sont généralement causées par des problèmes sous-jacents. La chaussée montréalaise connaît d'importants dommages prématurés et ceux-ci en forme de craquements (fatigue) et de nids de poules. Malheureusement, ces solutions n'abordent pas le vrai problème et ne sont que temporaires.

Cette recherche affiche un compte rendu des procédés de réparations et d'entretiens utilisées par la ville de Montréal, étudie leurs compétences, et montre comment les méthodes de conceptions et de dimensionnement actuelles engendrent des épaisseurs insuffisantes ainsi qu'un drainage inadéquat. L'objectif de cette thèse est de déterminer la raison pour laquelle la chaussée à Montréal se détériore prématurément relativement aux méthodes de conceptions, de réparations et d'entretiens utilisés. De plus, cette recherche contient une étude de cas approfondie de la condition des routes dans la Ville de Montréal, et ceci, en comparant les pratiques de constructions utilisées dans les provinces de Québec et de l'Ontario.

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Chapter One

Introduction

1.1 Importance of road pavements:

Road pavements play an important role in our daily life, as their performance can significantly affect our health, quality of life, and economy. The cracks, in poorly drained pavements, can allow contaminants to seep through into the underlying pavement layers to eventually reach and pollute the ground water; these rough pavement surfaces will result in loud traffic-noise emissions which can lead to serious discomfort for nearby residents. In addition, deteriorated pavement surfaces can cause serious traffic jams and accidents as drivers suddenly slow down or swerve around potholes and severe deteriorations. This will increase the green house gas emissions and the average fuel consumption per trip leading to higher transportation costs thus reducing the truck trade which is a main source of revenue for the province of Quebec (50 billion dollars in 1998). In fact, constructing and maintaining a good quality pavement can positively affect the citizen's quality of life, as fuel consumption, traffic jams and car damage are reduced. Finally, the local economy will improve significantly since truck transportation costs will decrease and temporary/seasonal jobs are created for maintenance, rehabilitation or even new construction projects.

1.2 Purpose and Scope:

Pavements in Montreal are in a state of extensive deterioration. Even when repaired, they tend to fail prematurely. Municipal officials tend to blame the lack of funding available for maintenance and repairs, whereas the provincial officials lay the blame on the harsh weather and poor subgrade material.

Previous research projects, such as those that were conducted on behalf of the Quebec Ministry of Transport (MTQ), had mainly focused on the use and the effect of new paving materials. In fact, many projects were determined on developing new asphalt mixtures for the surface course that would have a higher tensile strength or an increased resistance to thermal cracking.

The main purpose of this thesis is to determine how and why Montreal asphalt pavements fail so prematurely. This was ascertained by observing different modes of deteriorations at different locations and analyzing them in terms of design/repair strategies, materials used, and availability of drainage.

1.3 Outline of the thesis:

This thesis is divided into six chapters.

Chapter two, Design of urban roads, will discuss the strengths and weaknesses of the current pavement design theories and procedures as well as the main parameters of drainage design.

Chapter Three, Pavement deteriorations, describes the different layers in the pavement structure and discusses the cause of the various failure modes.

Chapter Four, Pavement maintenance and rehabilitation, discusses the importance of preventive maintenance and presents different maintenance, repair and rehabilitation techniques.

Chapter Five, Montreal Case study, analyses the different field observations in combination with the current design and construction procedures to determine the main reason for the premature failure of the pavements.

Chapter Six, Conclusion, provides a brief summary of the thesis and lists a series of conclusions.

Chapter Two

Design of urban roads

2.1 Pavement theory and design criteria

Road pavements are layered structures composed of varying physical characteristics and materials to support the applied loads and distribute them to the subgrade. They have been designed and constructed over the years using different procedures. The design methods are used to determine the thickness and materials in each of the layers for a given load, climate and service life.

Empirical design procedures, such as the American Association of State Highway and Transportation Officials (AASHTO) method of pavement design, are based on experience and extensive testing, whereas analytical procedures are used to determine the resistance and response of pavements to the applied loads in terms of calculated stresses, strains, and surface deflections (Canadian Strategic Highway Research Program (C-SHRP), Technical Brief No.23, 2002).

Current analytical or mechanistic design procedures consist of determining the response of the pavement structure as a multi-layer elastic system, where the materials are characterized by their modulus of elasticity and Poisson's ratio. These results are then calibrated using an empirical factor for local in-place conditions to achieve an economic and durable pavement, thus the name mechanistic-empirical (Croney and Croney, 1997). The multiple-layer elastic theory is based on the assumption that the layers are isotropic and homogeneous; the loads are assumed to be static, and the material behavior is assumed to be linear-elastic and follows Hooke's law.

The empirical design procedures are based on two main failure mechanisms derived from the earlier American Association of State Highway Officials (AASHO) road tests (1956-1961). They limit the horizontal tensile strain, ϵ_t, at the bottom of the surface layer to reduce fatigue cracks, and the vertical compressive strain, ϵ_c, on the top of the subgrade to limit permanent deformation or rutting (Huang, 1993).

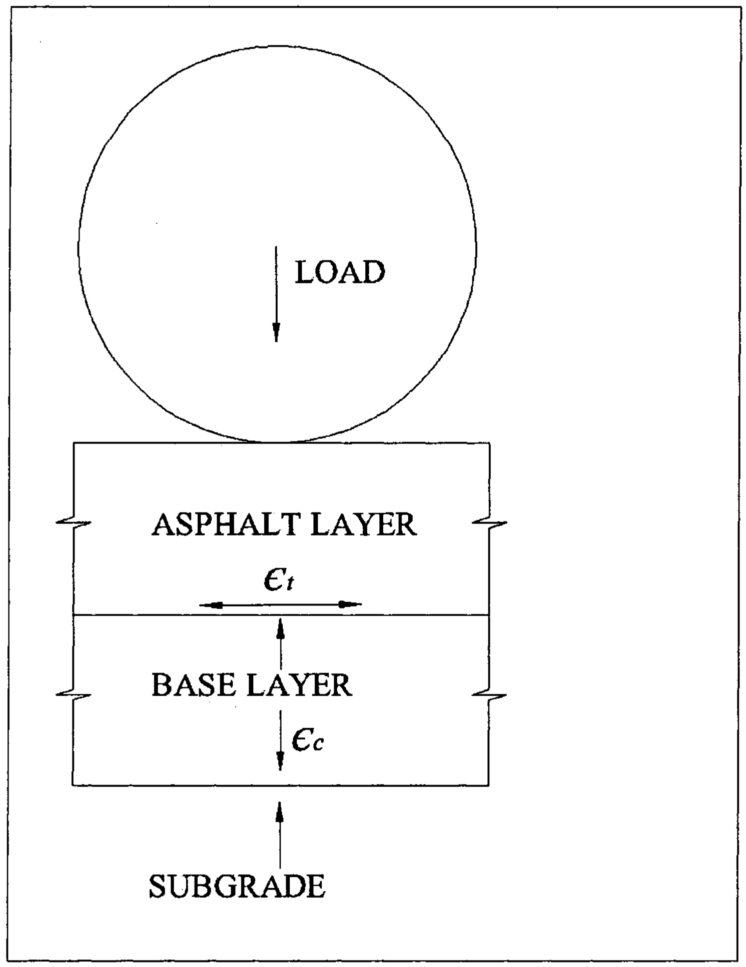

Figure 2.1: Horizontal and vertical strains

Empirical equations 2.1 and 2.2 enable the prediction of the maximum allowable number of load repetitions (N_f and N_d) before failure over the service life of the pavement.

The failure criterion for rutting is given by:

$$N_d = f_4 (\epsilon_c)^{-f_5} \quad \text{(Eq. 2.1)}$$

where N_d is the maximum allowable number of load repetitions before failure by rutting and ϵ_c is the vertical compressive strain at the top of the subgrade.

The constants f_4 and f_5 vary with different agencies as they are determined according to the local permissible maximum rut depth. The values for f_4 and f_5 as well as the maximum allowable rut depth, as recommended by specific agencies, are shown in Table 2.1.

The failure criterion for fatigue cracking results in the equation:

$$N_f = f_1 \times \epsilon_t^{-f_2} \times E_1^{-f_3} \quad \text{(Eq. 2.2)}$$

where N_f is the limiting number of load repetitions prior to fatigue cracking, ϵ_t is the limiting tensile strain at the bottom of the asphalt layer, and E_1 *is* the young modulus of the asphalt layer.

The factors f_1, f_2 and f_3 are specific to each agency and are determined from laboratory fatigue testing. They must be corrected to suit the local field conditions.

Agency	f_4	f_5	Rut depth (mm)
Asphalt Institute	1.365E-9	4.447	12.5
Shell (revised 1985)			
50% reliability	6.15E-7	4.0	
85% reliability	1.94E-7	4.0	
95% reliability	1.05E-7	4.0	
U.K. Transport and Road Research Lab (85% reliability)	6.18E-8	3.95	
Belgian Road Research Center	3.05E-9	4.35	10.2

Table 2.1: Subgrade strain criterion used by various agencies (Huang 1993).

The U.S Army Corps of Engineers (COE) has developed its own limiting subgrade strain criterion as:

$N_d = 10,000 \times (a/\epsilon_c)^{b}$ (Eq. 2.3)

Equation (2.3) is used to determine N_d, the maximum number of allowable load repetitions, before rutting occurs. This is achieved by limiting ϵ_c, the vertical compressive strain, at the top of the subgrade. The factors a and b vary according to the subgrade strength and are calculated using equations (2.4) and (2.5) and the modulus of the subgrade M_r in MPa.

$a = 0.000247 + 0.000245 \log [(145). (M_r)]$ (Eq. 2.4)

$b = 0.0658 [(145). (M_r)]^{0.559}$ (Eq. 2.5)

Figure 2.2 shows the different subgrade failure criteria recommended by the specific agencies.

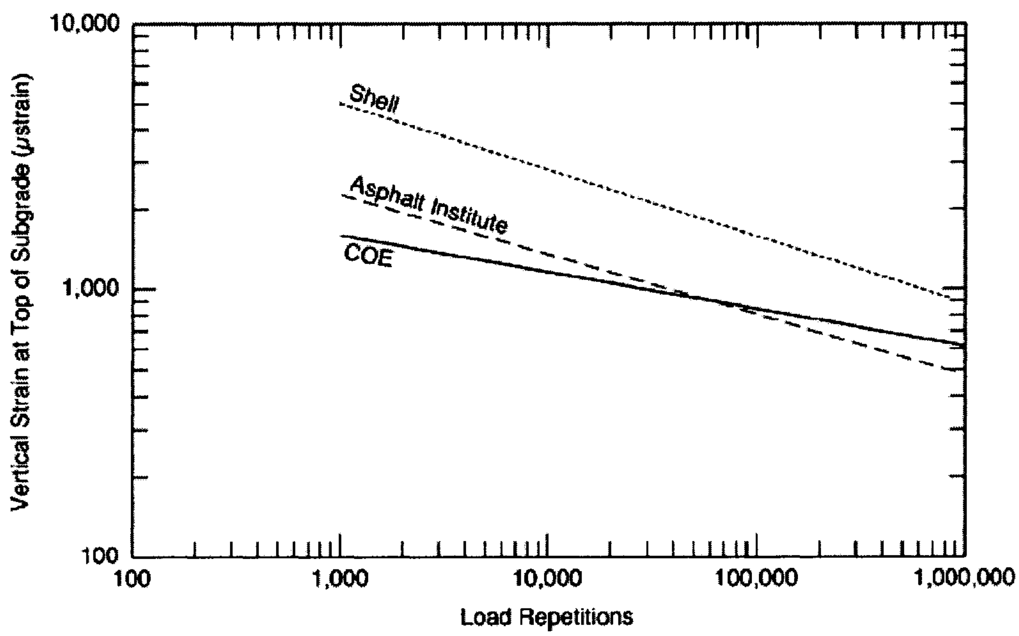

Figure 2.2: Subgrade failure criteria

(U.S Army Corps of Engineers, 2003)

2.2 Empirical and analytical design methods:

2.2.1 The AASHTO design procedures:

The AASHTO 1986 (National Cooperative Highway Research Program, NCHRP Synthesis 189, 1993) Design Guide uses an empirical approach for pavement design. It is based on the results of the AASHO (American Association of State Highway Officials- predecessor of AASHTO) road tests (1956-1961), which observed the performance of selected pavements of known thicknesses and material composition in identified environments (711mm frost depth and 864mm annual rainfall) and on a specific subgrade type (a poorly drained A6 soil as classified by AASTHO Soil Classification System M-145-91 (1995) shown in Table (2.4), with a CBR value ranging from 2 to 4). The subgrade soil properties as well as those of the base and subbase material properties are listed in Tables (2.2) and (2.3), respectively. The following empirical equation (Eq. 2.6) is used to predict the maximum allowable number of 80 kN (18 kips) Equivalent Single Axle Load (ESAL) W_{18}. It is a function of the layer thickness and properties, the resilient modulus of the subgrade soil (M_R), and the level of deterioration (Present Serviceability Index PSI).

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07 \qquad \text{(Eq. 2.6)}$$

where Z_R is the normal deviate for a given level of reliability, S_0 is the standard deviation, ΔPSI is the change in PSI over the service life of the pavement, M_R is the resilient modulus of the subgrade and SN is the structural number of the pavement structure. SN can be calculated using Eq. (2.7)

$SN = a_1.D_1 + a_2.D_2.m_2 + a_3.D_3.m_3$ (Eq. 2.7)

The structural number (SN) is used to determine the structural strength of the pavement. D_1, D_2 *and* D_3 are thicknesses of the surface, base and subbase layers, respectively. Each of these layers is assigned a layer coefficient, a_n, representing its structural capacity. The m_n factors are assigned only to the base and the sub-base layers as drainage coefficients.

The AASHTO design guide owes its success to simplicity and to the limited input requirements which result in an economical design. Unfortunately, like all empirical methods, the AASHTO procedures can result in weak pavements that can experience premature failure, if constructed in conditions outside of the range the test conditions.

Classification (AASHTO M-145) (AASHTO 1990)	A-6
Average values, borrow pit samples	
Max dry density (AASHO T-99-49) (kg/m^3)	1858
Optimum moisture content (%)	15
Liquid limit	29
Plasticity index	13
Grain size finer than (%):	
No. 200	81
0.02 mm	63
0.005 mm	42
Specific gravity	2.71
Average of construction tests:	
Density (% max. dry density)	97.7
Moisture content (%)	16
Constructed embankment tests	
Laboratory CBR, soaked	2–4
Field in-place CBR, spring	2–4
Modulus of subgrade reaction, *k* (MN/m^3)	12

Table 2.2: AASHO road test subgrade soil properties

(U.S Army Corps of Engineers, 2003)

Item	Subbase	Crushed stone base	Uncrushed gravel base
Aggregate gradation, % passing			
38.1 mm sieve	–	100	100
25.4 mm sieve	100	90	98
19.1 mm sieve	96	80	–
12.7 mm sieve	90	68	74
No. 4 sieve	71	50	49
No. 40 sieve	25	21	23
No. 200 sieve	7	11	9
Plasticity index, minus No. 40 material	–	–	3.5
Maximum dry density (kg/m^3)	2210	2227	2243
Field density (% max. dry density)	102	104	102

Table 2.3: AASHO road test base and subbase material

(U.S Army Corps of Engineers, 2003)

General Classification	Granular Materials (35% or less passing No. 200)							Silt-Clay Materials (More than 35% passing No. 200)			
Group classification	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5 A-7-6
Sieve analysis, % passing:											
No. 10 (200 mm)	50 max	...	...	...	...	...	...	...	...	...	...
No. 40 (425 µm)	30 max	50 max	51 min	...	...	...	...	...	...	...	...
No. 200 (75 µm)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing No. 40 (425 µm)											
Liquid limit		...	...	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	Stone Fragments Gravel and Sand		Fine Sand	Silty or Clayey Gravel and Sand				Silty Soils		Clayey Soils	
General rating as subgrade	Excellent to Good					Fair to Poor					

Table 2.4: AASHTO Soil Classification (AASHTO Designation M-145-91, 1995).

2.2.2 The Asphalt Institute Design method

The Asphalt Institute (A I) has adopted an empirical procedure for pavement design. It analyzes the pavement as a multilayer elastic system using the "CHEVRON N-Layer" program to determine the stresses, strains and deflections in the various layers of the pavement structure. The empirical part of the design is the relationship between the pavement response and deterioration (A I, 1981). The Asphalt Institute uses the equations Eq. 2.2 and Eq. 2.1 (defined earlier in Section 2.1) as failure criteria for fatigue cracking and permanent deformation where the values of f_1, f_2 *and* f_3 are 0.0796, 3.291 and 0.854, respectively. The values for f_4 *and* f_5 are shown in Table 2.1.

This method can be used to design new pavement structures, or to rehabilitate an existing one by measuring the total pavement deflection and back-calculating to determine the elastic modulus of each layer.

Although the stresses are determined analytically, the multilayer elastic theory remains a general approximation of the "real" situation. The underlying assumptions do not reflect "reality" as the applied loads are dynamic in nature; material behavior is anisotropic and visco-elastic rather than linear elastic. Furthermore, the design procedure does not account for cracking in the structure which will result in discontinuous strains along the depth of the pavement structure.

2.2.3 Numerical design - finite and distinct element analysis:

Finite element analysis provides a refined approach to pavement design. It can evaluate the pavement response under static or dynamic loads, for elastic, plastic and visco-elastic behavior of the materials involved. The reliability of the analysis results depends on the accuracy and validity of the assumptions made. Unfortunately, this complicated numerical method requires extensive testing for validation against "real" data. Moreover, the current finite element methods can only be used to design new pavement structures and do not allow for direct elastic modulus back-calculation (from measured surface deflections) for rehabilitation of existing structures.

Another computer assisted, analytical design procedure is the distinct element method which models deformations in granular materials resulting from sliding or rolling. It uses the grain size and distribution, stiffness, as well as the coefficient of friction and degree of compaction as input to determine displacements and forces between particles at any location within the pavement structure (Ullidtz, 1998). The disadvantage of this method is the extensive computing requirements and the processing time required for the technique to be effective and efficient.

2.3 Drainage design

Water in pavements exists in the form of free-water, moisture, or capillary water. Of these, only free water in the pavement layers can be drained by gravity flow.

Free water can enter the pavement either by surface infiltration through the joints and cracks, from the melting of ice lenses within the pavement layers or from below as the groundwater rises during thaw, or after a heavy rainfall. The ice lenses are formed above the water table level where the frost penetration reaches the capillary water in the soil that has risen by capillary suction.

If free water is not quickly removed, unbound granular layers, as well as the subgrade soils will experience a noticeable loss of strength/stiffness resulting in a weak pavement that will deform excessively and fail under normal traffic loads.

In cold climates, this problem is compounded by the expansion of water as it freezes, which will cause frost-heave in the pavement during winters and a reduction in the stiffness during thaw (spring). Furthermore, saturated layers can experience pumping of material under dynamic traffic loads resulting in high hydrodynamic pressures resulting in an eventual loss of support, or even stripping. An adequate subsurface drainage practice consists of laying a high permeability lateral drainage layer that discharges water in the longitudinal edge drains placed in ditches below the pavement structure. This is not the only possible combination as each drainage system can function separately. The longitudinal drains are used to discharge surface water infiltrating the pavement and to lower the existing water table, whereas the drainage layer is used to discard only the water due to surface infiltration. Combining the two can result in a drainage system which is faster, more durable, and easier to maintain. The choice of the drainage type and design is primarily influenced by the infiltration rate, the permeability of the materials used (base, subbase and subgrade), and by the design criteria selected. The surface infiltration rate is dependent mainly on the thickness of the surface water film and on the duration of that film on the surface of the pavement. The permeability of granular material used is closely related to the fine particle content in the aggregate gradation as shown by Barber and Sawyer (1952). According to NCHRP Synthesis on Highway Practice 96 (1982), the drainage design criteria are time-dependent and can be measured either by the time needed to drain a certain percentage of water in a saturated pavement, or by the inflow and outflow rates that will maintain the pavement base and subbase layers below the saturation level.

The gravity induced water movement is assumed to follow Darcy's law for laminar flow which is used for the design of the base and the subbase layers (Huang, 1993):

$$v = k \, . \, i \qquad \text{(Eq. 2.8)}$$

where v is the discharge velocity, k is the permeability of the layer and i is the hydraulic gradient.

$$Q = v \, . \, A \qquad \text{(Eq. 2.9)}$$

where Q is the flow rate and A is the cross-sectional area perpendicular to the direction of flow)

The lateral drainage layer can be a highly permeable base layer, or part of the subbase placed on top of the subgrade. The first choice results in a faster draining pavement and prevents pumping in the underlying layers but raises stability issues as no or little fines are present, and the coarse material is left unsupported. A common solution is to use a cement/asphalt-stabilized open-graded aggregate layer as a base layer to achieve increased pavement strength. In cases where the drainage layer is placed at the top of the subgrade, the permeability of the base and the subbase must be increased so that the discharge flow rate exceeds the infiltration rate (Huang, 1993)

Finally, a study conducted by NCHRP (1997) determined that many premature failures have been attributed to inadequate surface drainage. It was also noted that longitudinal drains do not improve the performance of the pavement but can double its durability. This is only true as long as the materials and the soil are permeable enough to allow the water to reach the drains.

Chapter Three

Pavement deterioration

3.1 Pavement structure:

Road pavements are designed to withstand dynamic traffic loads and to have adequate durability and serviceability during the service life. The complexity of this design does not lie within the structure of the pavement but in the thickness and the materials used in each layer. The layered structure of the flexible pavement typically consists of three main courses: the asphalt concrete wearing course (A.C), the base and/or subbase, and finally the subgrade (Hunter, 2000).

As stated previously, these layers are designed to transmit the stresses resulting from the traffic and distribute them to the subgrade. Pavements can be classified as flexible, semi-rigid or rigid in an increasing order of the modulus of elasticity of the material of construction, and the stiffness of the pavement which is directly affected by the thickness and properties of each of the layers. A higher modulus layer will have a higher load capacity, reducing the stresses applied to the subgrade for the same wheel load.

3.1.1 Asphalt concrete wearing course

In flexible pavements, the A.C layer is composed of a surface course (direct contact with tires) laid on top of a binder course. A tack coat will provide the necessary bond between the two layers (Hunter, 2000). The wearing course in semi-rigid pavements generally consists of a surface course laid directly on the concrete base course. The surface course is a high-stability A.C mixture with more asphalt content and finer aggregate gradation than the binder course. The design of the A.C mixture is guided by the need to attain acceptable resistance to permanent deformation, fatigue, low temperature cracking, stripping, skidding and finally to achieve adequate workability for the construction process and durability.

The design of the asphalt-aggregate mixture in the binder course can be done using the Superpave method, the Hveem method or the Marshall method. According to Zaniewski and Nelson (2003), the asphalt content and the density of the mixture are the two variables that control the design and performance of the pavement.

The density of the asphalt mix is dependent mainly on its air void content.

The latter is controlled by the asphalt content, compaction during construction, and most importantly, by the traffic induced compaction. A compaction-induced increase in density will not only reduce the permeability of the mixture, but also increase its shear resistance, thereby reducing deflections. Increasing the asphalt ratio, by filling up the air voids with bitumen, is another way to increase the density of the mix, but it will greatly decrease its shear strength making it prone to permanent deformations or ruts. In the end, the final surface must provide a smooth and safe ride with minimal noise under daily traffic conditions.

3.1.2 The Base course:

In flexible pavements, the base layer is composed of high-stability (angular), frost resisting, well graded crushed gravel, or granular material treated with cement, lime, or bitumen. In semi-rigid pavements, a reinforced or unreinforced concrete slab forms the base layer. In all pavement structures, the base course must provide the required stiffness for an adequate fatigue resistance while reducing the stresses being transmitted from the surface to the subgrade. In cases where the design yields a large value for the base layer thickness, it may be divided into two distinct layers: base and subbase layers. In this case, the subbase is generally made of inexpensive material, coarse graded aggregates (with less than 8% passing the No.200 sieve) (Roads and Transportation Association of Canada, 1971). These granular materials can also be treated with cement, lime or asphalt.

The main parameters in evaluating the base performance are the grading of the granular materials, the frost resistance of the aggregates, and the permeability of the final base structure so that the drainage of water is possible while preventing pumping of the material which weakens the pavement structure, and causes permanent deformations leading to failure.

3.1.3 Subgrade:

The subgrade is the foundation layer for the pavement structure. It may be the natural soil left behind from the construction cuts, or refilled material with better properties. The performance of the subgrade is evaluated through three main characteristics, the load bearing capacity, the moisture content, and the shrinkage/swelling properties of the soil. The subgrade must support all of the loads transmitted from the upper layers of the pavement structure and resist deformations under all conditions. The moisture content in the subgrade has a direct effect on the load bearing capacity as well as on the swelling/shrinkage of the soil. Poor drainage, high pavement permeability and high ground water level adversely affect the subbase moisture content leading to excessive swelling/ shrinking specially in cold climates where frost heave may occur along with excessive deformations.

The material used in the subgrade layer must be easy to compact so that no differential settlement can occur under heavy traffic loads, have adequate shear strength to resist permanent deformations under dry and wet conditions, and have excellent drainage properties to prevent pumping and the weakening of the subgrade materials.

The shear strength of the subgrade can be determined using different methods. The California Bearing Ratio (CBR) (Croney and Croney, 1997) is an empirical scale relating penetration to bearing strength. It compares the value obtained to that of a well graded crushed stone subgrade (0%-100%). Other methods are available such as the resilient modulus method, which is gaining popularity due to the consistency in its results, and the resistance value (R-value) method (Jones and Harvey, 2005).

3.2 Surface deteriorations

After and during construction, the pavement structure is subjected repetitively to heavy loadings, harsh weather and other external factors. These impede the performance of the pavement by making it rough, discontinuous and unsafe for traffic. Surface deteriorations can be separated in four categories: cracks, distortions, surface defects, and patches and potholes. They are called surface deteriorations because they manifest themselves at the surface but their root is normally in the underlying layers of the pavement structure. They are identified visually and can be graded under low, moderate or severe level of deterioration. These deteriorations must be closely investigated to determine the aggressive attacking factors so that proper repair plans can be developed and implemented. Table 3.1, shown at the end of this section, presents a summary of the main causes for some of these deteriorations, and proposes a selection of repair and maintenance procedures.

3.2.1 Cracking:

The surface of the pavement can exhibit the following patterns of cracking: fatigue cracks, thermal cracks, longitudinal cracks, block cracks, reflection cracks and slippage cracks.

3.2.1.1 Fatigue cracking (alligator):

Fatigue cracks are closely spaced, interconnected and angular shaped cracks. These cracks are load-associated and usually caused by the repetition of stresses on the pavement structure higher than those considered in the design (Roberts et al. 1996). Fatigue cracking can also be caused by poor quality control during construction resulting in an inadequate layer thickness. Inadequate drainage of the subsurface layers will aggravate this problem by saturating the base and the subgrade materials leading to a dramatic loss of strength. Repetitive heavy axle loads combined with a decrease in strength will cause the pavement structure to deform extensively, resulting in an increase in the tensile strains at the bottom of the A.C layer. This increase in strain will result in the propagation of micro-cracks from the bottom of the A.C layer up to the surface forming the alligator cracking.

3.2.1.2 Thermal cracking:

In cold climates, daily and seasonal temperature variations create tensile stresses in the asphalt surface layer due to the thermally induced shrinkage. Thermal cracking occurs when these stresses are higher than the tensile strength of the A.C mix. These cracks initiate at the top of the surface layer and spread towards the bottom layers. They are equally spaced and in the transverse direction of the road. Thermal cracking is controlled by the asphalt mixture properties. Low penetration, high viscosity, or high temperature susceptibility mixtures are subject to severe thermal cracking. This is caused by the thermoplastic nature of asphalt, where a dramatic increase in stiffness is observed as temperature drops.

3.2.1.3 Longitudinal cracking:

Longitudinal cracks run in the direction of traffic as they initiate at the top of the A.C layer and propagate downwards. They can occur at the joint between different mixtures, or at the edges of the wheel paths in a rutted pavement. Longitudinal cracks are caused by the residual stresses developed outside the rutted area which exceed the tensile strength of the mix. In cold climates, as the A.C layer gains stiffness, heavy axle loads applied in the path of the rut, in a pavements resting on a weak/wet subgrade, will experience high deformations. These high deformations combined with the temperature-induced increase in stiffness will crack the A.C layer along the edges of the rut.

Moreover, since the A.C layer has been laid in sections, the joints between adjacent batches have high air void content. This can lead to a decrease in density, reducing the tensile strength of the asphalt mix along the joint. In addition, this higher air void content will cause early age hardening of the mix in the joint area making it brittle. The asphalt at the joint will be brittle and it will have a lower tensile strength than the other areas of the A.C layer which will make it prone to longitudinal cracking.

3.2.1.4 Block cracking:

Block cracking is mostly observed in low traffic areas and in widely paved areas. When low traffic volume is not enough to densify the A.C layer, it is left with high air void content. This air content will oxidize and harden the mixture allowing it to exhibit thixotropic hardening resulting in cracking at low temperatures. In conclusion, block cracking is similar to thermal cracking with the only difference being the thixotropic hardening of the asphalt mix due to the low volume of traffic.

3.2.1.5 Reflective cracking:

Reflection cracks are cracks exhibited at the surface of the pavement due to discontinuities in an underlying layer. Reflection cracks will exist at the joints between underlying concrete slabs or at the location of older cracks in the binder course. The high residual strains at the tip of the old crack will rupture the newly laid asphalt layer at its base and move upward toward the surface of the pavement. If no preventive measures are taken to reduce reflection cracks, they can propagate to the surface at an average rate of one inch per year (Mamlouk and Zaniewski, 1998).

3.2.1.6 Slippage cracking:

When poor bond exists between the A.C surface course and the underlying layer, the function of the pavement in transmitting all the loads to the subgrade is stopped. Shearing forces from daily traffic loads will overstress the pavement's surface creating early U shaped cracks.

Repetitive loadings combined with cold and wet conditions will further aggravate the problem by dislodging material and creating a hole along the depth of the surface layer.

3.2.2 Distortion:

As stated earlier, pavement surfaces can undergo different types of deteriorations. When the surface layer exhibits deteriorations in the form of permanent deformations, they are called distortions. The distortions of the pavement surface can be categorized into rutting and shoving.

3.2.2.1 Rutting:

Rutting is characterized by the differential settlements in the wheel-path due to repetitive traffic loads. These surface distortions do not affect the serviceability of the pavement as much as the traffic safety. The uneven pavement surface can inhibit the lateral surface drainage as water fills the ruts. This will lead to a decrease in the skid resistance caused by the possibility of hydroplaning (Hunter, 2000). Rutting may be caused by poor compaction, a poorly designed mixture or an inadequate pavement design.

During construction of the pavement, if layers are not compacted to their optimal density, rutting will take place in the wheel-path due to the traffic induced compaction. Furthermore, lateral plastic flow will occur in the A.C layer when the loads are carried by the binder rather than by the aggregate structure. This will occur when the A.C mix contains either high asphalt content or low stability aggregate mixtures (more rounded aggregates with smooth surface than rough textured angular aggregates). In the case where pavements are poorly designed, the inadequate layer thickness will overstress the subgrade under daily traffic loads. This problem combined with insufficient drainage can lead to shear failure of the subgrade. These failures are accompanied by permanent deformations that will manifest as surface ruts.

3.2.2.2 Shoving and corrugations:

Shoving (wavy pattern) and corrugations (ripples) are localized distortions at the surface of the pavement that run perpendicular to traffic flow. They occur at locations where shearing forces are applied due to traffic braking, accelerating or negotiating hard turns (90°) (Huang, 1993). These distortions can be the result of plastic flow in the A.C layer due to the high asphalt content and low void ratio in the asphalt-aggregate mixture (low stiffness). The shearing forces displace the aggregates in the surface layer creating shoving or corrugation. Also, an inadequate thickness control during construction or an underestimated design can lead to overstressing the subgrade resulting in shoving, or corrugation. This problem can be compounded by improper subsurface drainage which will further weaken the subgrade.

3.2.3 Surface defects:

Pavement structures can also deteriorate as the asphalt binder breaks down or as aggregates are polished. The pavement surface defects such as ravelling, stripping and wear loss are summarily reviewed in the following sections.

3.2.3.1 Ravelling:

Ravelling is a progressive disintegration of the A.C surface layer that initiates at the surface of the pavement and progresses downward. It is characterized by the dislodging of aggregates from the surface course accompanied with a loss of the binder material (Huang, 1993).

The loss of bond between the asphalt coating and the aggregates can be caused by improper A.C mix design, insufficient compaction and inadequately graded aggregates.

When the A.C mix has low asphalt content and is combined with insufficient compaction during construction, the surface layer will have high air-void content. This will cause insufficient cohesion and premature aging of the asphalt binder resulting in a weak and brittle bond with the aggregates. A dust layer covering the aggregates surface can also weaken that bond (Roberts et al., 1996). If the binder film covering the aggregates is not thick enough, it will stick on the dust rather than on the aggregate.

Finally, the insufficient fines content in coarse-graded aggregate mixtures can cause severe ravelling on the pavement surface. The unstable aggregate mixture will segregate; coarse particles will then rest on each other, reducing their surface contact with the cement, thus weakening the bond strength.

3.2.3.2 Stripping:

Stripping is characterized by the failure of aggregates-cement bond in the A.C mix that initiates at the bottom of the A.C layer and progresses upward (Hunter, 2002). It can be reflected at the surface through rutting, ravelling or cracking and by the presence of partially coated aggregates in the A.C layer. Stripping is usually a moisture induced problem and can be accelerated by a weak aggregate-cement bond.

In poorly compacted pavements, excessive air voids in the A.C layer will make the asphalt permeable allowing ingress of water under traffic loads. As this layer becomes saturated due to inadequate subsurface drainage, high pore pressure develops within these voids under traffic loads rupturing the binder-aggregate bond. Additionally, in cold climates, the expansion of the water in the voids will create enough stress to initiate stripping.

In pavements where low permeability layers are placed on top of saturated high permeability layers, high pore pressure develops under traffic loads in the voids at the cement-aggregate interface, due to pumping leading to the failure of that bond. This phenomenon will set off stripping, eventually leading to the formation of pothole (Kandhal, 1992). When pumping is the main factor in initiating stripping, the pavement surface will first exhibit damage in the form of wet spots. After it has dried, the fine particles and unbound asphalt cement are left at the surface of the pavement in the form of white spots. These white spots can result in the formation of fatty areas caused by the high concentration of asphalt and fines and may lead to the formation of potholes.

Moreover, stripping can be caused by a weak aggregate-asphalt bond. That bond is greatly reduced if the aggregates have either high residual moisture content or dusty surfaces. According to Parker (1989), aggregates with high residual moisture will have more potential in initiating stripping. In the presence of water, dust and fine clay particles at the aggregate-asphalt interface will separate from the aggregate causing stripping of the pavement.

3.2.3.3 Wear Loss:

Wear loss is caused by the shearing forces generated from the tires at the surface of the A.C layer. It is characterized by the gradual removal and polishing of the aggregates at the pavement surface. In aged asphalt pavements, aggregates are easily removed from the oxidized A.C surface due to the shearing forces from traffic. This will create loose material that will act as abrasive and accelerate the wearing process. Ravelling will accelerate wear loss deteriorations because of the high presence of loose fine materials and aggregates.

Stiffler (1969) showed that the wear loss of the surface A.C layer is inversely proportional to the hardness of the abrasive and directly proportional to the loads applied.

3.2.4 Potholes and patches:

3.2.4.1 Potholes:

Potholes are small but fast growing, bowl-shaped holes that run form the surface of the A.C layer down to the base course (Roberts et al, 1996). They have vertical and sharp edges at the surface of the pavement and are wider at the bottom. The formation of potholes depends on the moisture content and the drainage of surface and subsurface layers.

Generally, as rain water percolates from the surface into the pavement, the underlying granular layers are softened and material is displaced. In cracked pavements, pumping of fines from the subbase course will further weaken the foundation of the structure. Under traffic loads or thaw, the weakened pavement structure will deform extensively resulting in localized surface depressions leading to the formation of fatigue cracks first, or directly potholes. In cold climates, during thaw, these cracked pieces of the A.C surface loose their bond with the surrounding material and are removed by the rolling action of tires over the pavement .The same phenomenon is observed in all pavements when subsurface layers are weakened by the rise of underground water level due to heavy rainfall, thaw or inadequate drainage.

Pothole formation is generally followed by ravelling due to the dislodging of the poorly bonded aggregates from the impact of the tires on the edges of the pothole at the A.C surface. Once ravelling initiates, combined with the pumping and the displacement of subsurface material next to the hole, the rapid growth of that pothole is inevitable, until proper repair and patching procedures are implemented.

3.2.4.2 Patches:

Patching techniques are used to repair most deterioration types and utility cuts. It consists of removing the defective portion of the pavement, fixing the source of the problem, and finally restoring the pavement layers. If the problem is not fixed, the previous deterioration will resurface after traffic is resumed. Even though patching is a repair technique, patched pavement surfaces are considered as deteriorations. Full depth and partial depth patching techniques are discussed in Chapter Four.

Poor bond with the surrounding pavement material can create new deteriorations such as shoving/corrugations, alligator cracking and slippage cracking. Additionally, water seeping into the opened joints reduces the strength of the underlying layers and can lead to rutting and ravelling at the edges of the patch.

Types of deteriorations	**Possible causes**	**Repair methods** (Roberts et al., 1996)	**Suggestions for prevention/ maintenance** (Mamlouk and Zaniewski, 1998)
Fatigue and alligator cracking	• Inadequate layer thickness • Weak/wet subbase	• Structural overlay • CIPR (to stabilize the weak layer, drainage improvement is required) • Reconstruction	• Thickness control during construction. • Stabilize and drain weak subgrade soil. • Implement regular drainage maintenance plan.
Thermal cracks	• Mixture properties (penetration, viscosity…)	• HIPR • Mill at crack depth and overlay	• Low temperature susceptibility asphalt mixture • Sealing cracks
Longitudinal cracks	• Age hardening • Low ductility mix • Differential settlement of a weak/wet subgrade	• Sealing cracks. • Structural overlay and improve subsurface drainage	• Adequate drainage of subgrade. • Adequate compaction of asphalt between lanes (reduce oxidative hardening).
Block cracking	• Thixotropic hardening	• HIPR • CIPR and overlay • Structural overlay	• Mix quality control. • 5% air void content after compaction.
Reflective cracks	• Discontinuities in the underlying layer	• CIPR and overlay • Mill all cracked layers and Overlay • Place a crack relief layer at old pavement surface before overlay.	• Saw-cut the asphalt overlay at PCC slab joint. • Implement seasonal maintenance plans to inspect, clean and seal all joints

Slippage cracks	• Failure of bond between layers	• Mill below slipping depth and overlay.	• Mill/ roughen the old pavement surface. • Control tack-coat thickness during construction
Rutting	• Inadequate compaction during lay down • High binder content in the mix • Wet subgrade • Inadequate layer thickness	• Drain and Stabilize weak/wet subbase • HIPR • Structural Overlay • CIPR and overlay • Reconstruction.	• Improve subsurface drainage. • Use high stability aggregate mixtures (angular, rough) • Stiffer asphalt mixture
Shoving & corrugations	• Plastic flow in the mix • Inadequate subsurface drainage	• HIPR. • Mill and overlay. • CIPR and overlay. • Structural overlay. • Reconstruction.	• Use angular aggregates and reduce binder content. • Ensure adequate bond between layers is achieved
Ravelling	• Low binder content • Poor compaction, high air void content • Insufficient aggregate coating, dusty aggregates • Unstable aggregate structure	• HIPR • Mill and overlay	• Use clean, dry aggregates. • Ensure adequate coating off Aggregates. • Ensure adequate compaction during construction.
Stripping	• Poor subsurface drainage, Pumping • Water ingress in permeable layers • Water expansion at the aggregate-binder interface • Moist or dusty aggregates	• Remove and replace stripped layers • CIPR and overlay	• Use of anti-stripping additives. • Use of clean, rough aggregates. • Adequate layer compaction during construction • Ensure adequate drainage of subsurface layers.

Potholes	• Permeable layers, flow of unbound material and inadequate surface drainage • Poor subsurface drainage, movements in the weak subbase. • Freeze – thaw in the thaw cycle. • Stripping. • Fatigue cracking	• Patching • Structural overlay • CIPR and overlay	• Use bound material in subsurface layers. • Provide sufficient subgrade drainage. • Impermeable pavement surface layers. • Ensure adequate surface drainage. • Limit stripping and fatigue cracking

Table 3.1: Summary; cause, repair and maintenance of selected failures.

3.3 Concrete base course deteriorations

The performance of a concrete base course in semi-rigid or composite pavements depends on the condition of the subbase, subgrade, and the A.C surface course. Each of these inter-related layers can either cause the failure of the pavement or help in supporting another weakened layer. The main function of the layers below the base course is to provide full support for the concrete slab as a main load carrying layer in the pavement structure. It is necessary to state that the steel reinforcement is used in slabs, only to control temperature cracking and not to increase the flexural strength of the pavement. The concrete base can undergo different deteriorations which will be reflected at the surface of the A.C layer reducing the overall serviceability of the pavement structure.

3.3.1 Physical deterioration

3.3.1.1 Plastic shrinkage cracks:

Plastic shrinkage cracks are caused by the capillary tension developed in the pore structure as the concrete dries. They can appear at the surface of the slab as map cracks, or parallel cracks forming a 45° with the slab corners. These cracks initiate at the surface and propagate through the entire depth of the slab. Concrete mixes with a high water/cement ratio, poorly vibrated, or not properly cured, will have a higher probability of experiencing plastic shrinkage cracking.

3.3.1.2 Transverse and corner cracking:

The concrete slabs forming the base course may crack in a direction perpendicular to the traffic flow, especially near the end, and diagonally across the corners. These cracks are usually caused either by the repetitive application of heavy concentrated wheel loads, or by the movements of the subgrade from the wetting/drying and insufficient compaction of the underlying layers. The subgrade is more likely to settle near the edges and corners of the slab because of the warping stresses resulting in curling of the slab. That section must act as a bridge to the traffic loads over the consolidated area. That loss of support creates tensile stresses, considerably exceeding the tensile strength at the concrete slab resulting in transverse or corner cracking (Sparkes, 1952).

Differential settlement under the slab may be caused by the changing moisture content in the subgrade, plastic flow of clay under heavy traffic loads, frost heave, or structural failure of the soil.

3.3.1.3 Temperature and shrinkage cracks:

Temperature cracks appear when a temperature gradient occurs along the depth of the concrete slab, whereas shrinkage cracks occur due to the long term drying of the concrete. In both cases, a state of stress equilibrium is created between the top (tension) and bottom (compression) surface of the slab. If the stress exceeds the strength of the concrete within the slab, map cracking occurs. These cracks usually start at the surface of the slab and move downward.

3.3.1.4 Splitting, spalling and scaling of the surface:

Splitting, spalling and scaling of the slab are a result of water expansion, due to freezing, in the concrete. In poorly designed concrete mixtures, high water cement ratio, as well as inadequate vibration will result in high capillary channels and large entrapped air voids. If these voids are saturated, and temperatures are below freezing, water expansion in the voids is inevitable. This high porosity and permeability of the concrete will result in splitting the concrete leading to surface cracks. It is necessary to state that the freezing temperature of water decreases with the size of these voids, and that splitting of concrete is only a problem in completely saturated concrete slabs.

Spalling will occur when non-frost resisting aggregates are used. These highly porous aggregates absorb moisture and expand as water inside their pores freezes. This expansion will generate tensile stresses cracking the cement paste and causing local pop-outs or spalling of the surface (Huang, 1993).

Scaling of the concrete surface occurs when internal stresses develop inside the concrete slab. As deicing chemicals are used, their concentration decreases with increasing depth. This will cause different sections of the slab to freeze at different times, resulting in scaling of the surface.

3.3.2 Chemical deterioration:

3.3.2.1 Acid attacks

The strength of a concrete base slab after the 28-days period is determined by the chemical composition of the hardened concrete cement. The calcium compounds, especially the calcium silicate hydrate, are responsible for the concrete strength. In the presence of acids, these compounds will react forming calcium salts. The solubility rate of the residual salts will determine the deterioration rate of the concrete slab.

Slabs experiencing acid attacks will exhibit loss of hardened surface cement paste as the concrete is destroyed layer by layer. Magnesium and Ammonium salts will result in the same mode of deterioration as their equivalent acids whereas soft water will dissolve the calcium compounds. The extent of the deterioration is relative to the amount of substance available and on the extent of the exposure area.

3.3.2.2 Sulfate attacks

Concrete slabs are highly susceptible to sulfate attacks from the soil or from seeping water. Sulfate ions react with the hardened cement structure to form ettringite. The latter will expand excessively in the presence of water/moisture resulting in an irregular cracking pattern and spalling of the concrete. This will cause disintegration of the concrete slab. The permeability of the concrete will determine the extent of the deterioration since it limits the penetration depth of the sulfate ions into the slab.

3.3.2.3 Alkali-silica and alkali-carbonate reaction:

The alkaline properties of the concrete cement (lime saturated pores) may react with silica containing aggregates and result in parallel, and map cracking patterns. These cracking patterns, which initiate at the surface of the slab and progress downward, result from the expansion of the resulting alkali-silicate compound and lead to the complete disintegration of the concrete. The failure of the concrete slab may also be caused by an alkali-carbonate reaction, where dolomite containing carbonate minerals react with the alkaline solutions present in the concrete. The resulting compounds, magnesium hydroxide and calcium carbonate, will cause expansive pressures leading to surface map cracking that precedes the complete disintegration of the slab (Huang, 1993). Both of these deteriorations are limited by the amount of reactive in the aggregates and hydroxyl ions present in the concrete pore structure.

3.4 Drainage of pavements and frost action:

Adequate drainage of pavement structures is critical in improving the performance and durability of the structure while reducing the maintenance costs. Previous sections have clearly shown that, although they act quite differently, surface and subsurface water are both responsible for most pavement deteriorations.

Drainage systems are used to intercept, collect and drain all water (surface and subsurface) from and below the pavement structure to improve the strength of the layers and reduce frost damage. In general, the performance of a drainage system is measured by the time required to drain all pavement layers from a certain quantity of water. The drainage of the pavements can therefore be separated into surface and subsurface drainage.

3.4.1 Surface drainage:

Surface drainage is the process by which pavement structures eliminate the surface runoffs. This can be achieved by using draining accessories or by channeling surface water to the nearby gutters or ditches. To have a safe ride, pavement surfaces must be free from water accumulation that will increase the amount of spray and splash, decrease the skid resistance of the pavement, and potentially cause hydroplaning (NCHRP 1998).

Hydroplaning will occur once enough water has accumulated and a thick water film has developed at the surface causing the tires to loose contact with the pavement. Inadequate lateral surface drainage can be caused by poor geometric design (cross-slope), elongated flow path due to lack/loss of A.C surface macro-texture or surface deteriorations, such as rutting, ravelling, and localized depressions. Moreover, slotted drains and grate inlets must be properly spaced so that surface drainage is optimized.

Surface water can also damage pavements if it reaches the underlying granular layers. The deposit of fine soil particles from surface water seepage into joints and cracks will impede the draining capacities of the subsurface granular layers causing them to saturate and lose strength. Stripping and potholing can then initiate in the A.C surface layer.

3.4.2 Subsurface drainage:

Water can penetrate the subsurface layers of the pavement from below, as groundwater table rises during thaw or heavy rainfall, from side ditches, or from above as water percolates through the permeable surface layers (cracks and joints increase permeability). If not drained effectively, subsurface water can cause major deteriorations in the pavement layers (NCHRP 1982).

The flow of water through the pavement layers may create a hydrostatic head resulting in an upheaval and a failure of the structure, while the saturation of the granular layers, as well as an increase in subgrade moisture will result in extensive deformations under heavy traffic loads. Both cases will result in major cracking, leading to the disintegration of the pavement surface (and concrete slab in case of semi-rigid pavements). Inadequate subsurface drainage or silted drains play a major role in these deteriorations.

Furthermore, pavements built on clay subgrades can undergo severe deformations cased by moisture movements in the soil. These movements are generated by an osmosis phenomenon, where water moves from wetter sections to dryer sections of the clay subgrade. This will either cause the clay under the pavement to swell or to settle depending on the surrounding environmental factors. Moisture movement cannot be controlled by subsurface drainage. It can, however, be limited by installing a water-proofing, or a geocomposite membrane in the subgrade to stop these movements (Christopher et al., 1999).

3.4.3 Frost action:

Water in the subgrade can rise higher than the ground-water table by capillary suction. In cold areas, where frost reaches high depth within the soil, ice lenses are formed within the pavement structure. They are caused by the capillary movements of water from beneath the pavement structure (Croney and Croney, 1997). The expansion of the capillary moisture, or the undrained free-water within the structure will cause the pavement to heave and crack. Such large deformations can cause heavy damage to the subsurface drainage systems which must be placed below the frost line. Silts and silty clay soils are the most susceptible soils to freezing (Highway Research Board, 1951).

Chapter Four

Pavement maintenance and rehabilitation

4.1 Maintenance and repair in asphalt pavements

4.1.1 Preventive maintenance:

After construction, pavement structures are continuously subjected to traffic loads and harsh environment which reduce their performance. When pavements are managed in a "build and forget" strategy, their performance deteriorates and the rehabilitation cost increases dramatically with time as shown in Figure 4.1 (CPM, 2005). The rate of deterioration occurring in these pavements can be effectively inhibited by implementing cost/time effective preventive maintenance plans. These plans are intended to preserve the structural integrity of the pavement by the timely application of selected treatments prior to the apparition of significant distress. According to NCHRP (1979), savings of $ 100,000 per mile (asphalt thickness of 3.75 inches) were achieved when an overlay was placed as a preventive measure prior to failure instead of waiting until the pavements has failed.

Preventive maintenance treatments are discussed below and include fog seals, chip seals, and slurry seals.

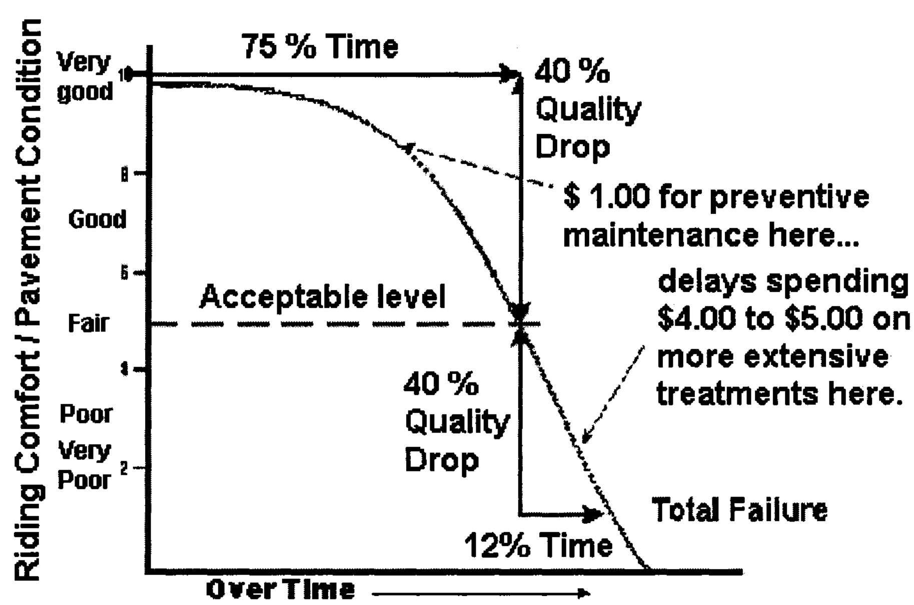

Figure 4.1: Effect of deferred maintenance (CPM, 2005)

4.1.1.1 Fog seals:

Fog seal treatments are applied on low traffic volume roads where the asphalt at the pavement surface has aged. This is achieved by evenly spraying a diluted solution of bitumen emulsion on the surface of the pavement. It is necessary to ensure that the fog seal emulsion must have the adequate stability and viscosity to be able to penetrate the pavement surface. According to Roberts et al. (1996), a slow setting cationic emulsion diluted in an equal amount of water is suitable as a fog seal solution.

As water evaporates from the solution, the emulsified asphalt sets up on the old surface. If excess asphalt is applied, a thin and smooth asphalt layer will form resulting in the loss of skid resistance.

Fog seals are used mainly to enrich the oxidized asphalt surface, seal fine surface cracks and inhibit the progress of ravelling. Their success can be attributed to their relatively low cost and to their ability to temporarily postpone surface treatments (2 years).

4.1.1.2 Slurry seals and micro-surfacing :

Slurry seal is a cold surface treatment composed of a bitumen emulsion, an additive and well-graded fine aggregates, mixed in a specialized truck on site. It is applied continuously, in a single layer, on low and moderate speed roads as a final impervious coating to the pavement surface. The thickness of the coating is nearly equal to the size of the maximum aggregate used. Roberts et al. (1996) distinguishes three types of slurry seals, depending on their thickness and the purpose of their use.

Type I slurry is used to seal fine surface cracks and voids to prevent the ingress of water and aggressive elements into the pavement structure. Consequently, a fine aggregate gradation is required, resulting in a thin coat at the rejuvenated pavement surface as shown in Table 4.1 (Roberts et al., 1996). The seal is applied as a single coat continuously along the pavement surface. Its small thickness makes it usable only on low speed roads.

Type II slurry is used on pavements experiencing ravelling and wear loss. The mixture is composed mainly of angular shaped/polish-resistant aggregates (maximum aggregate size=6mm) and a quick setting emulsion for early reopening to traffic. The resulting layer will be thicker, more durable, and will withstand higher traffic volume than the previously discussed type I seal.

Type of Slurry	I	II	III
General Use	Crack Filling and Fine Seal	General Seal on Medium Textured Surfaces	1st or 2nd Application, 2 Course Slurry on Highly Textured Surfaces
Sieve Size	% Passing	% Passing	% Passing
3/8 in.	-	100	100
#4	100	90-100	70-90
#8	90-100	65-90	45-70
#16	65-90	45-70	28-50
#30	40-60	30-50	19-34
#50	25-42	18-30	12-25
#100	15-30	10-21	7-18
#200	10-20	5-15	5-15
Residual Asphalt Content, % by Weight of Aggregate	10-16	7.5-13.5	6.5-12
Application Rate, lb/yd^2	6-10	10-15	15 or more

Table 4.1: Aggregate gradation requirements for slurry seals

(Roberts et. al, 1996)

Type III slurry is used when pavement surfaces have severely deteriorated. The larger aggregates used in this type will result in a thicker coat which will fill minor surface depressions and ruts. The main function of this layer is to seal, rejuvenate and re-profile the old pavement surface, while a denser layer with adequate skid resistance is laid on top. This practice is known as micro-surfacing. Micro-surfacing is the technological "next step" to slurry seals. The addition of polymers to quick setting emulsions has allowed for multiple layer application. Micro-surfacing treatments are cost/time effective treatments, applied to high speed and heavy traffic roads, to provide a new and even finish to the pavement surface. The new surface has an increased resistance to shoving, rutting and thermal cracking, while providing an improved skid resistance.

Micro-surfacing treatments will alter neither the structural nor the drainage capacity of the pavement. They will seal cracks, restore a poorly patched area, help in transverse surface leveling or fill moderate depth ruts (38mm). They owe their success to their low cost, time efficiency and to their ability to postpone the need for rehabilitation for up to seven years.

4.1.1.3 Chip seals or surface dressing:

Chip seals or surface dressing treatments consist of spraying a bituminous binder on a prepared surface, followed by a thin layer of graded aggregates. Aggregates are then rolled into the binder layer forming a thin and protective new wearing surface for the pavement. Chip seals can be applied in single or multi-layer treatments. They are used to fill surface cracks and raveled areas, increase skid resistance of a worn surface and provide an impermeable layer to protect the pavement against ingress of moisture and aggressive elements.

According to Hunter (2000), the success of the surface treatment is mainly related to the hardness of the old pavement surface, the binder type and the type and condition of the chippings.

During their service life, treated surfaces will experience heavy traffic loads which will contribute to further embedding of the aggregates in the old pavement surface. This can lead to loss of texture and bleeding of the pavement surface. In cases, where not enough embedment is achieved, stripping is initiated. The rate of embedment is controlled by traffic speed and volume, the size of the aggregates and by the hardness of the previous surface. More binder must be used on hard surfaces to account for the lack of embedment, whereas less binder must be used on soft areas to prevent fatting up the surface.

Binders can be modified bitumen, cutback bitumen or most commonly used bitumen emulsions, depending on the traffic volume, cost/time and environmental conditions as shown in Figure 4.2.

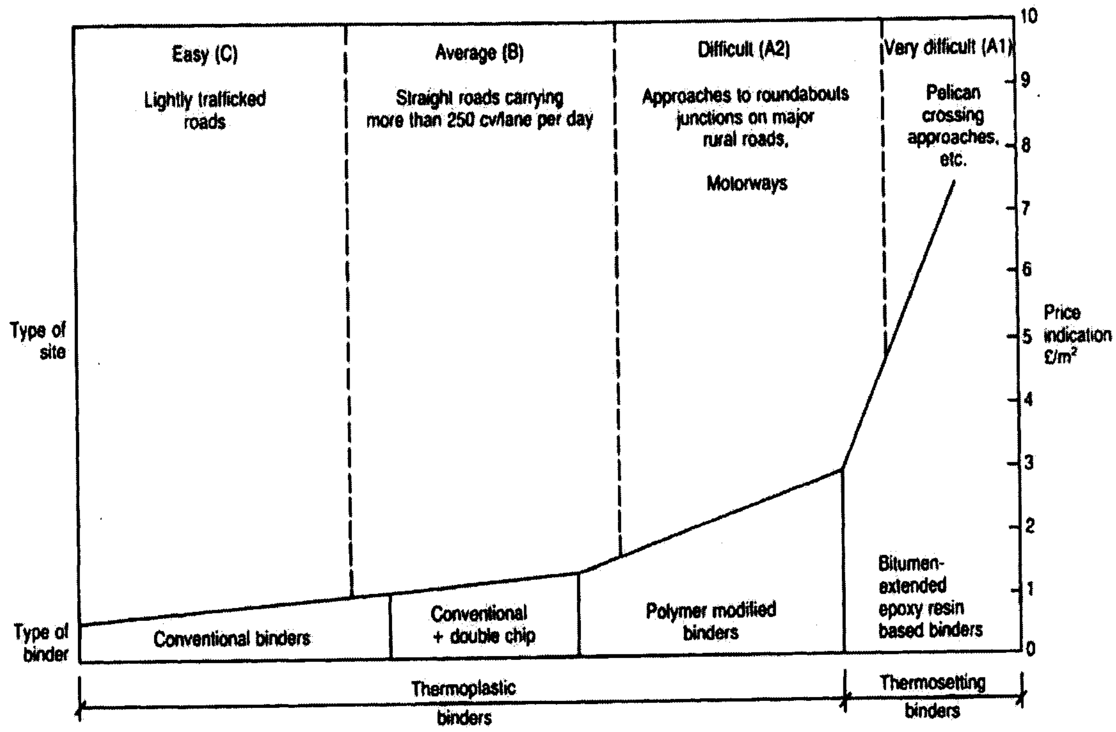

Figure 4.2: Binder specifications (Hunter, 2000)

The viscosity of the binder selected is critical to its performance. It will determine the coating capacity of the binder as well as its resistance to prolonged periods of low temperatures. Moreover, the binder used must be compatible with the aggregates to achieve adequate adhesion. If testing shows incompatibility, precoating the aggregates is mandatory to achieve an acceptable bond capacity. Dusty aggregates will also have a poor bond with the binder which can result in a loss of surface texture and more extensive ravelling. Also, the use of flat and elongated aggregates will lead to bleeding or flushing.

Finally, chip seals treatments can be applied using different techniques, having different characteristics, depending on the stresses applied. Type I slurry seals can be used as a preparation layer for chip seal treatments in order to provide a better bond with the old pavement surface.

4.1.1.4 Crack treatments:

Cracks in pavements need to be sealed to prevent ingress of water, accumulation of sand and dirt, or even plant growth. Sealing procedures consist of: ***1***: routing the crack in order to provide adequate sealing depth, ***2***: cleaning it so that an acceptable bond develops with the sealant, and ***3***: pouring a hot crack sealant up to below the surface to prevent overflowing. Overflowing the crack may reduce the skid resistance over the sealed area since the sealant material offers relatively no surface texture. Sealing cracks must be implemented at moderate temperatures, immediately after the crack has developed. Typical candidates are longitudinal, transverse, block, and reflection cracks.

The sealing material must be able to resist all tensile and compressive forces generated from traffic loads and temperature variations. According to Roberts et al. (1996), rubberized asphalt crack sealers have been found to be very adequate. A more extensive study on developing performance-based specifications and selecting of bituminous crack sealants is available in Masson and Lacasse (1998).

4.1.2 Repair techniques:

4.1.2.1 Patches :

Patching is the most common technique used for temporarily repairing a localized pavement distress. It generally consists of removing the deteriorated material, fixing the cause of the problem and refilling with new or recycled paving materials. Depending on the severity of the distress, one can distinguish between full depth and partial depth patching. Partial depth patching is used to fix problems arising from a poor bond between the asphalt surface layer and the underlying layers, whereas full depth patching is used to correct deficiencies in the subsurface layers. The service life of the patchwork is dependent on the environmental conditions present at the time of the work and on the patching techniques used. Even if patches may last for years, they are still considered to be temporary solution until a later rehabilitation of the pavement structure.

4.1.2.1.1 Permanent Hot-Mix Asphalt (HMA) patch:

This full depth patching technique is widely used for repairing fatigue cracking, utility cuts, and potholes. Unfortunately, it can only be performed when temperatures are warm and the weather is dry so that the underlying layers and materials may be compacted to their optimal dry density.

According to Stevens (1985), a permanent HMA patch consists of cutting a polygon shaped area at a minimal distance of 300mm around the localized deterioration. Rectangular shaped patches have proven to be more durable than regular polygon shaped because of better corner (90°) adhesion and easier compaction.

The face of the cut must be vertical so that adequate bonding and interlock is achieved with the pavement materials. The next step will be to remove all damaged material including saturated layers until an intact layer is reached. This will be a good time to assess the subsurface drainage capacity and to undertake necessary corrective measures.

The newly reached layer is compacted, waterproofed, and the damaged subgrade material is replaced by a suitable granular fill which will be compacted to near its optimal density (*95% modified proctor*). Waterproofing the newly compacted base layer before refilling with hot mix asphalt can improve the performance of the patch. A tack coat must be applied to all vertical sides and to the bottom of the hole to achieve the required bond with the new HMA material. The latter is then placed in layers of 100mm thickness and compacted from the center towards the edges until the old surface is reached. The final patch surface must have an adequate density and a slight crown extending over the pavement surface so that further densification under regular traffic loads will result in a level surface.

Finally, applying a 200 mm wide chip seal at the edges of the patch to prevent water seepage through the joints will extend the life expectancy of the patch.

4.1.2.1.2 Temporary pothole patching:

Pavements suffering excessive pothole deteriorations may need emergency repairs under adverse climatic conditions. These are repetitive short term treatments, expensive in the long run, that are necessary until a more appropriate and permanent solution can be implemented. Different methods can be used depending on the money, material and equipment available.

A value engineering study, conducted by the U.S Corps of Engineers, classified five emergency repair methods in three groups (Table 4.2). It concluded that Group 2 methods (3 & 4) were the most economical repair methods that resulted in an adequate service life of more than 12 months. Method 5 was more expensive but resulted in a longer average service life.

Methods 1 and 2 belonging to group 1 are the most expensive and short lived off all three groups. They must only be used in extreme cases where all other repair techniques are rendered ineffective. The initial varying parameters of this study were the degree/method of compaction, the cleanliness of the hole, and the straightness of the cuts, if any.

4.1.2.2 Non-structural overlays:

Non-structural overlays are thin (<50mm thick) surface overlays that are used to reduce roughness, restore skid resistance and correct minor surface deteriorations such as rutting and environmentally induced cracking. This type of treatment can be applied on low volume traffic roads to rejuvenate the brittle and aged asphalt pavement surface, or on heavy and high speed traffic roads. In the latter case, the purpose of the overlay is to resist rutting, polishing and wear loss which is achieved by the use of high stability angular aggregates in a durable asphalt mixture. Thin overlays must be placed on sealed, waterproofed and leveled surfaces to have a satisfactory performance.

Type I slurry mix can be used as the preparation layer. The tack coat interface will help in achieving the necessary bond strength between the old surface and the new overlay. Special care must be taken when choosing the appropriate aggregate size for the overlay thickness. This treatment can only be applied under strict supervision, in warm and dry weather so that all materials are compacted to their optimal dry density before the asphalt mix reaches its cessation temperature.

Thin overlays do not contribute to the structural capacity of the pavement structure; this explains the need for structural overlays.

Method No	Description of method	Tons in place per shift (7.5 hrs)	Life of patch (months)	Annual cost per ton ($)
1	Fill hole in one lift with mixture, and compact by hitting the patch with the back of the shovel. No effort made to clean or shape the hole. No tacking of the exposed surface of the hole.	18.0	1	307.68
2	Same as method 1 but the compaction is made with the tire of a dump truck.	12.0	2	190.80
3	Shape the area to be patched with an axe and sledge, remove lose asphalt with mattock, sweep the area clean, tack the exposed surfaces of patch area, shovel in material and level with lute. Compact with wacker and seal edges.	6.0	7	63.29
4	Same as method No 3 except a pup roller is used for compaction.	7.0	7	61.41
5	Same as method No 4 , except the area to be patched is shaped using a pavement breaker	7.0	7	65.22

Table 4.2: Temporary patching procedures

(Reproduced from U.S Army Corps of Engineers, 1981)

4.2 Rehabilitation of asphalt pavements:

4.2.1 Structural overlays:

Structural overlays are thick asphalt overlays used as a rehabilitation technique to a weak pavement. Their increased thickness amplifies the pavement structural capacity and can be used to correct surface deteriorations, improve skid resistance and protect the underlying pavement structure from water ingress and further deterioration.

The thickness of the overlay is directly related to the condition of the existing pavement structure. It can be determined using any of the following methods: a component analysis, a mechanistic analysis, a deflection analysis, or using a recipe design (Finn, and Monismith, 1984).

In a component analysis, the thicknesses of the different layers existing in the pavement structure are expressed in function of an equivalent HMA thickness. This is done by assigning a coefficient to each layer, often determined from previous experiences or laboratory testing. A new total thickness is then determined accounting for the changes in the traffic loads, subgrade strength and the condition of the existing pavement. This new design is then compared to the equivalent thickness of the existing pavement so that the overlay thickness can be determined. The structural number of the pavement, SN, is given by

$$SN = f\,(p_t, W_t, S, R) \qquad \text{(Eq. 4.1)}$$

where SN = Structural number

p_t = Terminal serviceability index

W_t = Total 80 kN ESAL's

S = Subgrade support

R = Regional factor

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \quad \text{(Eq. 4.2)}$$

where $D_{1,2,3}$ = thickness of surface, base and subbase layers, respectively

$a_{1,2,3}$ = layer coefficients

Pavement Layer	Layer Coefficient
Surface Course	
▪ Plant mix (high stability)	0.44*
▪ Sand Asphalt	0.40
Base Course	
▪ Crushed Stone	0.14*
▪ Sandy Gravel	0.07
▪ Bituminous-Treated	
o Coarse-Graded	0.34
o Sand Asphalt	0.30
Subbase Course	
▪ Sandy Gravel	0.11*
▪ Sand or Sandy Clay	0.07

* Established from AASHO Road Test data

Table 4.3: Layer coefficients (Hunter, 2000)

The mechanistic analysis evaluates the elastic and viscoelastic properties of the existing pavement material under varying environmental and loading conditions. It requires simple and inexpensive testing (Marshall Test ASTM D 1559-60T) resulting in vast amounts of data that will allow a qualitative evaluation of the mix performance. Unfortunately, this method uses old, empirical relationships and laboratory testing that are neither representative of the in-situ performance and compaction, nor of the fundamental properties of the mixture.

The deflection analysis uses surface deflection measurements (FWD) to determine the effective strength of the existing pavement structure. "Limited deflection" criteria will help the designer determine the new required thickness for estimated future traffic loads. This is possible, since the deflection amount of a pavement under a known load can be calculated. This method does not account for the properties of the individual layer, but analyses the pavement as a complete structure. This will lead to biased results that can vary depending on the environmental conditions and the subgrade properties.

The design recipe uses the past experience and the local know-how to determine an adequate overlay mixture and thickness. In some instances, this method can yield durable and cost-effective solutions. However, this approach cannot evaluate the in-situ performance variations, nor clearly define the characteristics of the materials used in the mix which suggests the need for quantifiable design methods.

4.2.2 Portland cement concrete (base-course):

Concrete slabs in pavements are generally used for their strength, durability, and their ease of maintenance. When built and jointed correctly, they will last as long as they are maintained. Unlike asphalt concrete which ages, cracks and weakens with time, Portland cement concrete (PCC) becomes stronger as the cement hydrates with the availability of moisture.

Most deteriorations in the PCC are caused either by aggressive elements or moisture variations (movements) in the subgrade from water percolating through joints and cracks. A waterproofing seal coat can be applied at the surface of the slabs to seal the existing cracks before the asphalt course is laid. Joints must be inspected, cleaned, sawn and sealed as soon as they fail to prevent accumulation of incompressible material and water penetration.

As water percolates through the underlying pavement layers, subbase and/or subgrade material will be displaced or even pumped out under traffic loads creating gaps under the base course thus weakening the PCC slab. In addition, expansion of the slab into debris filled joints will result in high stresses that will shatter and further deteriorate the joint.

When the required inspections are deferred, extensive cracking and disintegration of the slab may occur, leading to the creation of reflective cracks in the asphalt surface course. At this point, rehabilitation of the base course is required. These methods are more expensive and labor intensive than the regular joint maintenance and are dependent on the extent and severity of the deteriorations.

When deteriorations are localized and caused by gaps in the underlying layers, a bituminous mixture can be injected under the slab to stabilize the subbase/subgrade materials and to provide a uniform support for the base layer.

In some cases, an extra granular layer can be inserted between the asphalt surface course and the concrete slab to strengthen the surface course making it more resistant to reflective cracking. Both methods have limited use and are not cost-efficient.

Cracking or breaking the concrete is commonly used on heavily patched (from utility cuts) or spalled but structurally sound concrete slabs to minimize reflective cracking. A weight is dropped on the pavement causing tight cracks along the depth of the slab creating different, independent and relatively small pieces (maximum area: $1m^2$). The load transfer will still be achieved by aggregate interlock at the surface of the thin cracks, while inhibiting any horizontal movement. The cracked concrete must be firmly seated into the underlying subbase layer to provide full lateral support and reduce vertical deflections. This will result in a strong and more stable but highly permeable base for a structural overlay. The seating of the concrete must be achieved using heavy rubber-tired instead steel-wheeled rollers to prevent bridging over low spots and to minimize reflection cracks.

Rubblizing the concrete base course is performed on heavily deteriorated (ASR, freeze-thaw damage) concrete slabs to minimize reflective cracking. When the slab has lost most of its structural capacity and cannot retain its integrity it can be broken into small angular aggregates with a maximum width of 200 mm. This process can weaken the structure since the broken concrete is not the main load carrying layer in the structure and cannot bridge over weak spots anymore. A structural asphalt overlay is required to provide the required load carrying capacity and to inhibit rutting. The design of the asphalt overlay thickness will follow regular pavement design procedures with a granular base course in place of a concrete slab.

These cracking, breaking, and rubblizing operations can be a perfect opportunity for the engineer to check and improve subsurface drainage. This will result in a better performing and a more durable pavement structure.

Figure 4.3: Rubbilized pavement

4.2.3 Recycling of pavements:

Presently, as environmental issues are raised everyday, the need for sustainable development and technology has become a major concern. The recycling of pavement material is a positive step towards achieving that goal while leading to noticeable cost savings (Table 4.3, Nichols, 1998) .According to Kandhal (1994), no statistically significant difference was found in the in-situ material properties and performance between virgin and recycled pavements. Other studies have shown that recycled pavements can in some cases perform even better than virgin pavements in terms of aging, cracking, deformation and overall durability (moisture damage).

Proportion of reclaimed material (%)	Estimated saving	Actual saving (from completed project)
	Proportional cost of new asphalt (%)	
20	11	10
50	24	24
75	36	29

Table 4.4: Potential and actual cost savings from pavement recycling (Nichols, 1998).

Recycling of pavements is performed when maintenance has been deferred and a cost/time efficient rehabilitation technique is needed. As pavements are milled, granular material are collected, treated and stored for recycling and reuse. The reclaimed asphalt pavement material (RAP) can be used as an addition to a regular HMA mix, as an aggregate in cold mix asphalt or even as a granular base course. RAP is generally used in in-plant (off-site) recycling, for hot or cold asphalt mixtures, and requires transportation, crushing, grading and stocking before processing and use. Unless structural rehabilitation is required, in-plant recycling may be found quite expensive due to the high heating costs and to the extra transportation and stocking involved. The use of in-place or in-situ recycling techniques may prove to be more economical. Hot and cold in-place recycling techniques are discussed below.

4.2.3.1 Hot in-place recycling (HIPR):

Hot in-place recycling of pavements is carried out on oxidized asphalt surfaces where ravelling and thin surface cracks have occurred. It can also be effective in restoring skid resistance and correcting small surface depressions such as minor ruts. Experience has shown that in Quebec, up to 20% savings in construction costs were achieved when HIPR was carried out instead of conventional rehabilitation techniques (Roads and Transportation Association of Canada, 1983). HIPR can be performed using surface recycling, surface repaving or surface remixing (Dedens, 2003). Surface recycling involves heating and scarifying the top 25 mm of the surface followed by the application of an adequate overlay thickness as a final riding course in a multiple pass treatment. Surface repaving is a single pass procedure that consists of removing, rejuvenating and replacing the top 25-50 mm of the pavement surface as a leveling course before applying a thin HMA overlay (25mm) as a final riding course. Compaction of the overlay must follow immediately after lay down to ensure a monolithic bond between the new and the recycled layer. Surface remixing involves removing and rejuvenating up to 50 mm of the pavement surface mixed with new aggregates to improve stability or virgin hot-mix to modify the viscosity and the penetration of the final mixture before it is laid down. In all three methods, the overlay is compacted using the standard compaction equipment.

HIPR can only be applied during dry and warm weather to achieve an adequate bond with the surrounding material. According to Roads and Transportation Association of Canada (1983), HIPR mixtures have lower ductility and may experience early transverse thermal cracking. These cracks will require regular maintenance as they are sawed, cleaned, and sealed.

4.2.3.2 Cold in-place recycling (CIPR):

CIPR is a durable and cost saving surface rehabilitation technique that offers a possibility for application during wet conditions (no stagnant water and no rainfall) and relatively mild temperatures (when temperatures are below 10°C early cohesion must be achieved) (Lee 1998). A study performed by McKeen et al. (1998) showed savings up to 2.05$/m^2 in construction and maintenance costs when using CIPR instead of mill and overlay techniques in a study performed in New Mexico.

As the pavement is milled to a certain depth (50-150mm), rap material is crushed, graded and mixed on site with a binding agent (a regular or modified emulsion or foamed asphalt) and a liquid rejuvenator. The environmental conditions such as moisture, temperature and possibility of rainfall must be monitored to modify the viscosity of the binder accordingly and allow time for the emulsion to set.

The final mix is placed, compacted and sealed as a stabilized base course on which a chip seal or a hot mix asphalt overlay is laid as a final riding course. It is important for the engineer to ensure that surface water does not seep into the underlying layers and that adequate subsurface drainage is provided to achieve the desired performance and durability.

CIPR has proven effective in reducing transverse thermal cracking in cold regions and slowing the propagation of reflective cracks in asphalt pavements. Fatigue cracks resulting from inadequate design (weak/thin layers) can be rehabilitated using CIPR as long as proper subsurface drainage is provided. Special CIPR techniques have also been effectively used to rehabilitate pavements that have deteriorated due to either an unstable aggregate structure or a poor bituminous mixture such as ravelling, potholes, rutting, shoving, and slippage cracks where the addition of corrective mineral aggregates may be necessary to achieve the desired binder content and aggregate stability (Lee, 1998). According to Cross and Du (1998), the use of quick-lime slurry as an additive in the mix will help improve the resistance to rutting and moisture-induced damage, besides increasing the tensile strength of the final mixture.

Chapter Five

Montreal Roads-Case Study

5.1 Field observations

Road pavements in Montreal exhibit different types of deterioration, normally two winters after construction. The 4000 km of paved roads managed by the City of Montreal, are at an advanced state of deterioration and any parts repaired have failed almost immediately after repair. This section will review the different modes of pavement deterioration that were recorded over some selected roads in Montreal, as well as their specific characteristics. The streets were selected depending on their strategic location (Table 5.1), as well as on the type, frequency and speed of the operating traffic. The Côte Des Neiges (CDN), Côte Sainte-Catherine, Des Pins Avenue, and Parc Avenue were chosen for their importance as effective linkages and for their high traffic volumes at moderate speeds. Furthermore, St-Catherine Street represents a high volume slow traffic road with small to medium local delivery trucks and buses. Table 5.1 shows the distance covered on each street that was surveyed, as well as the number of potholes compared to the other types of deterioration. These deteriorations included fatigue, thermal and longitudinal cracks, ravelling and patches. The average numbers of deteriorations per kilometer were noted and are presented in Table 5.2 to determine the level of reliability of the study. The results shown in Table 5.1 confirm that potholes are the main type of pavement deterioration in the City of Montreal.

Street surveyed	Distance covered (m)	Number of Potholes	Number of other forms of deteriorations
Cotes des Neiges boulevard	700	10	13
Des Pins Avenue	900	11	9
Ste-Catherine Street	1200	14	10
Du Parc Avenue	900	8	7
Cote Sainte-Catherine	300	7	9
		Total	
ALL	3700	50	48

Table 5.1: Location and number of pavement deteriorations

Street surveyed	Average of potholes/deteriorations per km	
	Potholes/km	Other forms of deteriorations/km
Cotes des Neiges boulevard	14.28	18.57
Des Pins Avenue	12.22	10
Ste-Catherine Street	11.66	8.33
Du Parc Avenue	8.88	7.77
Cote Sainte-Catherine	23.33	30
Averages of the Total	13.51	12.97
Mean of the Averages	14.07	14.93

Table 5.2: Averages per kilometer

A majority of the potholes encountered were in the asphalt surface layer with depths ranging between 30 to 100 millimeters from the top of the pavement. These potholes were caused mainly by fatigue of the asphalt surface, or stripping within the asphalt concrete (A.C) layer, while there was a clear evidence of debonding between the different asphalt layers which explained the shallow potholes. The pothole shown in Figure 5.1 was taken on St-Catherine Street on the corner of Du Fort Street during the spring of 2005. It is 40 mm deep, with loose aggregates at its base and presents signs of debonding from the underlying layer. This suggests that stripping was initiated at the bottom of the pothole resulting in weakening of the pavement structure, and causing it to fail by overloading leading to the formation of potholes.

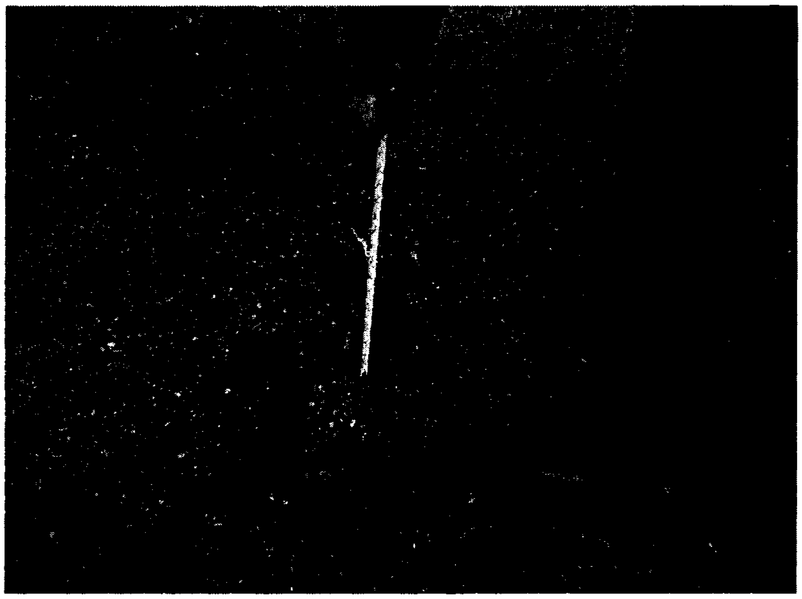

Figure 5. 1 : 40 mm deep pothole

(St-Catherine Street, corner of Du Fort Street, May 2005)

Deeper potholes (around 150 mm in depth) were also recorded over previously patched areas and randomly in low speed traffic lanes (trucks and buses). They presented signs of moisture and extensive fine particles at the pothole bottom, which suggests pumping from saturated underlying layers. The pumping of fine particles from the subbase, or the subgrade, can result in material displacement within the unbound layers of the pavement leading to a loss of support, and to the eventual formation of potholes. Figure 5.2 was taken in the slow traffic lane of Parc Avenue during the summer of 2005. It shows a 140 mm deep pothole which reached the concrete base. The presence of extensive fine particles at its bottom and clear signs of moisture implies that pumping from the saturated layers below the concrete base had taken place prior to the formation of the pothole. Furthermore, the high pore pressure resulting from the pumping caused stripping in the asphalt surface layer and permanent deformations in the subgrade (or subbase) layers resulting in the loss of support of the pavement structure, thus the formation of the deep pothole.

It should be noted that even though potholes are generally defined as bowl-shaped holes that are raveled at the edges, most of them were angular shaped and surrounded by severe alligator cracks. The angular shape indicates that the asphalt piece was removed after the alligator cracks were formed.

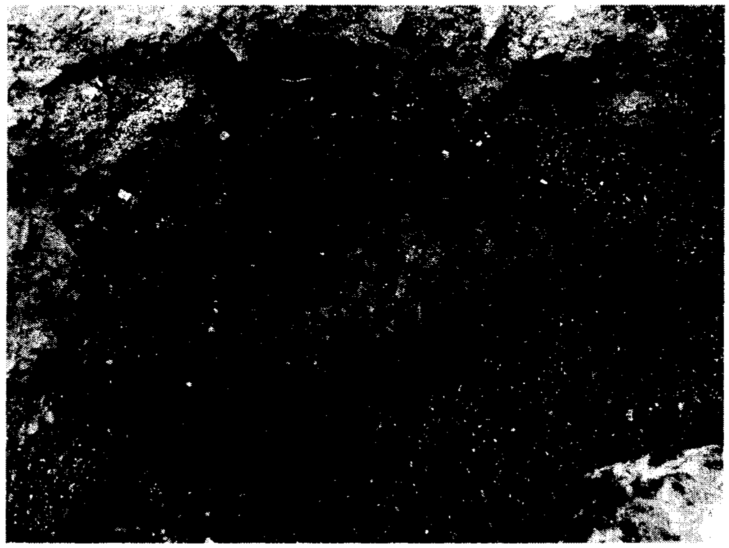

Figure 5. 2: 140 mm deep pothole

(Parc Avenue, corner of Duluth Street, June 2005)

The small angular pothole shown in Figure 5.3 was taken on the Cotes des Neiges Boulevard during the month of May in the year 2005. The presence of connected fatigue (alligator) cracks in its surroundings demonstrates that it occurred after cracking had taken place. In fact, as the cracked asphalt pieces loose their bond with the surrounding pavement material, they can be easily dislodged by the repetitive action of the tires rolling over them. Once the first cracked piece is removed, a small angular pothole is formed. The latter can then rapidly grow as the other cracked pieces will follow resulting in a wide but shallow angular pothole as shown in figure 5.4.

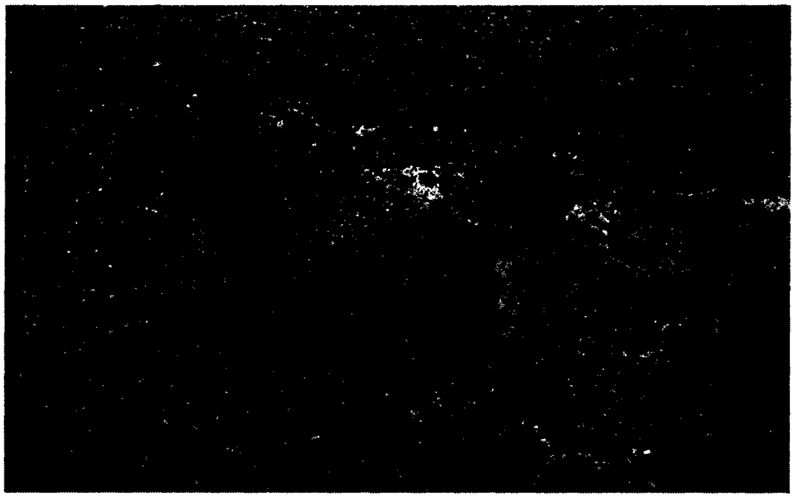

Figure 5. 3 : Small, angular pothole

(Cotes des Neiges Boulevard, corner of Belvedere Street, May 2005)

Figure 5. 4 : Wide and shallow angular pothole

(Cotes des Neiges Boulevard, corner of Decelles, May 2005)

The pothole in Figure 5.4 is more than 1200 mm wide and was also taken on Cotes des Neiges Boulevard during the month of May 2005. The alligator cracks present at the bottom suggest that the pavement strength at this section was decreased as its thickness was reduced after the pothole had formed. If it is not adequately repaired, this pothole will keep on growing deeper as successive layers will exhibit fatigue cracks and be removed by the tire action, resulting in a deep and severe pothole.

Finally, it is necessary to state that, in cold regions, potholes appear during the thaw period when the ice lenses formed within the pavement layers melt, thereby weakening the pavement structure. Currently, the changing weather and environmental conditions (warmer winters) are causing repetitive cycles of freezing and thawing within the pavement layers during a single winter season. This can explain the extensive deterioration problems in the pavement layers resulting in the excessive formation of potholes.

In an attempt to repair these deteriorations, the City of Montreal has adopted a selection of temporary quick-fix solutions which have been performing poorly and are proven to be expensive in the long run as discussed previously in Chapter Four. Out of the total number of patches observed (23 patches), nearly 70% were temporary (16), whereas only 30% were permanent repairs (7). All temporary patches showed different signs of distress, mainly debonding from the surrounding material and differential settlement caused by insufficient compaction. These failures were expected since the hole was not reshaped, or carefully cleaned and the asphalt mixture fill was inadequately compacted. Furthermore, these patching methods are emergency repairs only, performed during the wet season. They are intended to last until the environmental conditions (moisture and temperature) during the next summer or fall become adequate for permanent repairs. Figure 5.5, taken on St-Catherine Street in June of 2005, shows a series of old (1-2 years) and more recent (dark colour) temporary patches which exhibit cracking, debonding and potholes. This photograph explains the repair philosophy of the City of Montreal, as temporary repairs are used and considered to be "permanent", not needing any further action. The more recent patching jobs were performed during the spring of 2005 to repair sections of the older (light colour) patches that were deteriorating. In addition, these temporary repairs were selective since some cracks and potholes remain untreated, which contradicts the purpose of patching in sealing the surface of the pavement, against the ingress of water and aggressive elements, and in providing a smooth, safe riding surface. The present practice is unacceptable since it does not improve the performance or the durability of road pavements, while incurring significant costs to the City and thereby to the tax payer.

Figure 5. 5: Temporary patches

(St-Catherine Street, corner of St-Marc, June 2005)

Most of the permanent hot-mix asphalt patches that were observed had minor edge cracks and poor corner bonding caused by the lack of a 90° corner angle. Only a small percentage of these had settled due to poor compaction, or thawing. These problems can be further compounded by inadequate drainage, as surface water seeps through the cracks, reaches and weakens the underlying layers, resulting in excessive surface deformations. In cold climates, the expansion of water as it freezes to ice within the asphalt surface, will result in widening of the cracks and lead to the failure of bond with the surrounding material. The photographs of adequate and failed patches in the road surface are shown in Figures 5.6 and 5.7, respectively. The permanent HMA patch shown in Figure 5.6 was taken on St-Catherine Street in June of 2005 and is an example of a good performance patch. This is attributed to the straightness of the cuts and the 90° corner angles, which ensure adequate bonding and interlock with the surrounding material, and the degree of compaction which results in a smooth and level surface. Moreover, the failed patch in Figure 5.7, taken on Des Pins Avenue on the corner of Durocher Street in May of 2005, shows signs of shoving and excessive settlement. These deteriorations can be explained by the failure of the bond between the patching materials and the surrounding pavement. The weak bond could have been anticipated due to the wavy pattern and the round corners at the edges of the cut combined with the material movement due to the thermal variations.

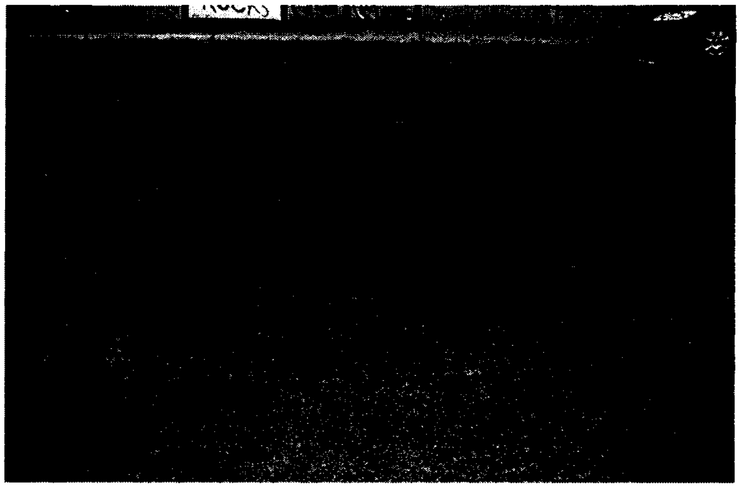

Figure 5. 6 : Adequate permanent patch

(St-Catherine Street, corner of Mackay Street, June 2005)

The resulting wide edge cracks will allow surface water infiltration into the asphalt layer, thereby weakening or rupturing the bond with the underlying layer and overstressing the pavement leading to excessive permanent deformations.

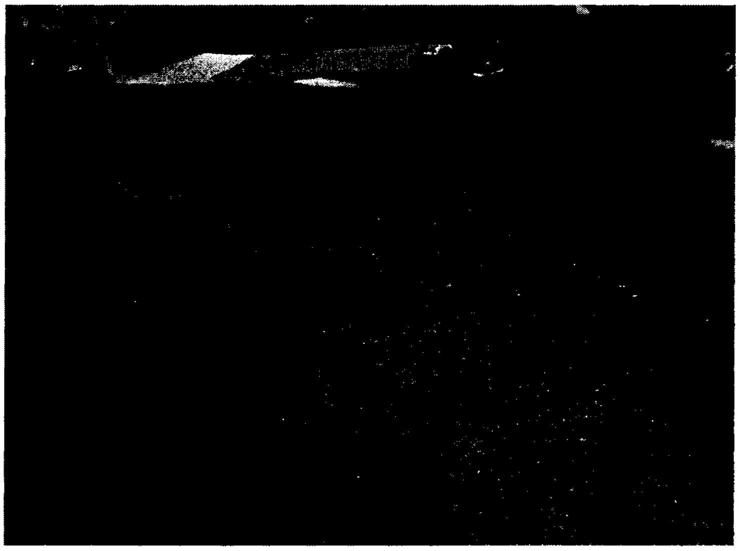

Figure 5. 7: Failed permanent patch

(Des Pins Avenue, corner of Durocher Street, May of 2005)

Overall, permanent hot mix asphalt patches in Montreal have demonstrated good performance as they present lower levels of deteriorations and an increased service life in comparison with the temporary patches. This success is mainly due to the fact that all of the deteriorated materials were removed and the problem was fixed prior to patching, whereas temporary repairs are performed as an emergency response to quickly seal and temporarily restore the asphalt surface.

Another type of deterioration which was frequently observed is fatigue cracking of the asphalt pavement surface. These cracks were present mainly along the unsealed cracks (Figure 5.8), or at the edges of the failed patches and were caused usually by the infiltration of surface water into the subsurface layers resulting in a loss of stiffness and permanent material displacement. Fatigue cracks were also found in the ruts, near potholes and bus stops. This suggests that either the applied loads are higher than those assumed during the design phase, or that the constructed pavement thickness is inadequate in providing the necessary resistance to the applied loads. The fatigue cracks can also precede the formation of potholes, as shown in Figure 5.8 and discussed earlier in this section. They are usually caused by a loss of strength in any of the pavement layers, or even in the subgrade.

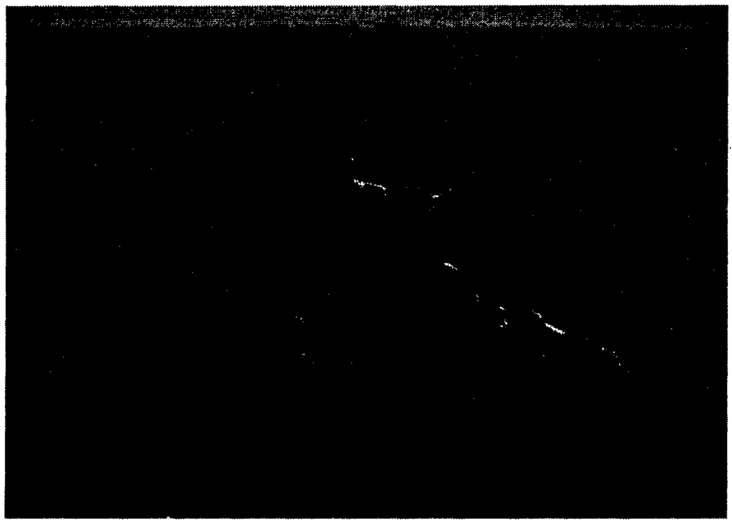

Figure 5. 8 : Alligator cracking along transverse crack leading to formation of potholes

(Cotes des Neiges Boulevard, corner of Rockledge Court, June 2005).

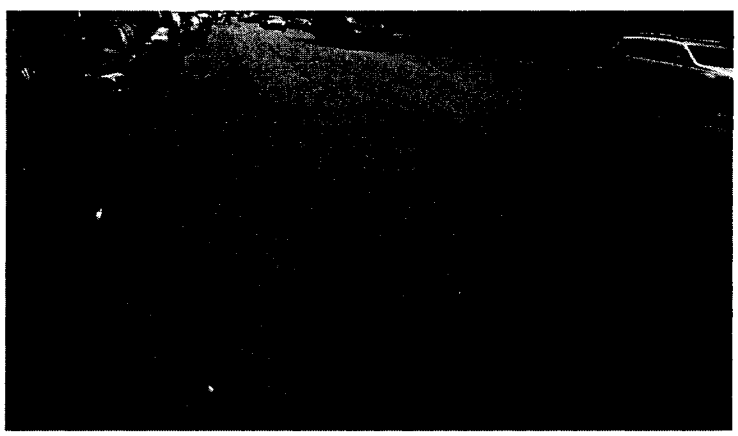

Figure 5. 9 : Fatigue cracking in ruts

(St- Catherine Street, corner of Pierce Street, June of 2005)

The photograph in Figure 5.9, taken on St- Catherine Street at the corner of Pierce Street in June of 2005, shows extensive alligator cracking in the rutted wheel path. As inadequate drainage caused the underlying layers to saturate, loose stiffness and weaken, heavy traffic loads caused shear failure in the subgrade leading to the creation of ruts. The repetitive traffic load on the weakened pavement resulted in the creation of alligator cracks within these ruts.

Additionally, all previously described deteriorations can be moisture-induced. The rate of deterioration is closely dependent on the permeability of the asphalt layer and the overall drainage capability of the pavement structure. Other observed deteriorations such as thermal cracks, reflective cracks, longitudinal cracks, as well as wear loss (shown in Appendix A) also increase the permeability of the pavement surface, allowing the surface water to infiltrate at a higher rate into the subsurface layers; this is a factor in the premature deterioration, shorter service life and much lower overall performance of Montreal City roads. The presence of heaved and settled sections are generally indicative of the freezing and thawing of the subsurface water within the pavement layers, requiring a review of the drainage requirements.

Wear loss (Figure 5.10) and longitudinal cracks (Figure 5.11) were mainly at the edges of the pavement where adequate compaction of the pavement layers could not be achieved and the required density was not attained due to the proximity of the sidewalk. This resulted in high air void content in the asphalt concrete which led to early age-hardening and brittle behavior of the asphalt surface.

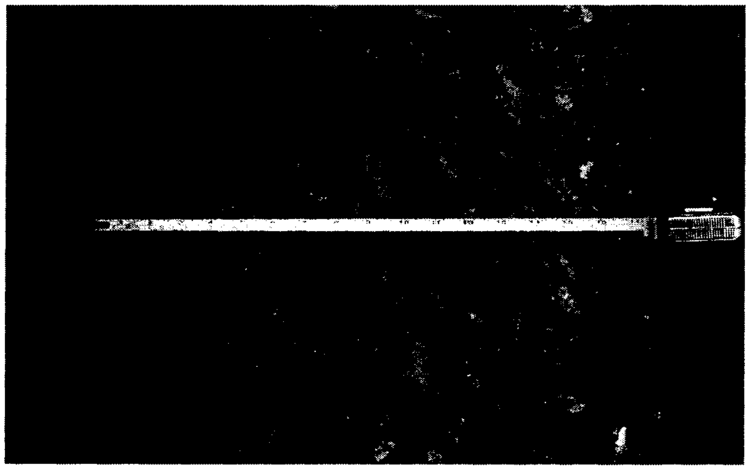

Figure 5. 10: Wear loss at the edges

(Cote Sainte- Catherine Street, corner of McNider Avenue, June of 2005)

Figure 5. 11: Longitudinal edge crack

(Des Pins Avenue, facing McGill gymnasium, May of 2005)

Figure 5.10, taken on Cote Sainte- Catherine Street in June of 2005, shows wear loss at the edges of the pavement, 45 mm away from the sidewalk. As mentioned earlier, the adequate compaction of the asphalt surface could not be achieved due to the proximity of the sidewalk. This resulted in a high air void content, and a decrease in the density, which lead to the early age-hardening and brittle behavior of the asphalt mixture. Under repetitive traffic, the shearing forces generated from the tire action on the brittle pavement surface removed the asphalt binder and the fine granules, creating loose abrading material that polished the remaining aggregates resulting in wear loss of the asphalt surface.

Figure 5.11 shows another type of deterioration that is similar to wear loss in terms of cause but exhibits itself differently, through longitudinal cracking. The longitudinal edge cracks that are shown in this photograph were taken on Des Pins Avenue in May of 2005. Like in the previous photograph (Figure 5.10), the closeness of the sidewalk interfered with the compaction of the pavement surface, resulting in a high air void content, thus a lower density and a more brittle (i.e. stiffer) behavior of the laid asphalt mixture. The relative decrease in density of the mixture, as well as its early age-hardening in comparison with the adjacent section, caused a significant decrease in its tensile strength. Under regular traffic loads, the differential deflections of the same pavement surface between the two adjacent sections, caused by the difference in stiffness, combined with the lower tensile strength, cracked the pavement longitudinally, parallel to the sidewalk and in the direction of traffic.

Finally, the overall performance of the pavement can also be affected by the adequacy or inadequacy of the repair methods used and by the efficiency of the maintenance plan and the quality control exercised.

In general, the City of Montreal uses the mill and asphalt overlay method to rehabilitate the road surface. This rehabilitation technique was closely observed during the beginning of May 2005, when it was being performed on a section of Sherbrooke Street, and in November of 2005 on a section of Cotes des Neiges Boulevard. Firstly, this technique is inadequate in repairing most of the observed deteriorations, such as the potholes, fatigue and reflective cracking as they normally reappear shortly after the first or second winter. This is caused mainly by the high residual strains at the tip of the old crack that will lead to the rupture of the asphalt overlay at its base and move upward toward the surface of the pavement (at a rate of 25 mm [one inch] per year). Secondly, the asphalt overlay procedure must be performed during the dry season (summer or early fall), which was not the case on Cotes des Neiges Boulevard, to achieve a strong bond between the new surface and the old / milled one. Another problem that was noted during the application of the mill and overlay technique is the poor quality control. Initially, not all of the cracked material was milled and the final surface was not leveled as shown in Figures 5.12 and 5.13. The photograph in Figure 5.12 was taken on Sherbrooke Street at the corner of Crescent Street in May of 2005. It can be clearly seen that the pothole was not leveled and the cracked surface was neither sealed nor removed prior to the application of the tack coat.

Figure 5. 12: Cracked asphalt surface after milling and prior to overlay

(Sherbrooke Street, corner of Crescent Street, May 2005)

Furthermore, the asphalt surrounding the manhole in Figure 5.13 had previously settled due to poor compaction, and shows signs of extensive cracking. The direct application of the overlay on top of this section, without removing the deteriorated materials, will only waste the available funds. In fact, this process will hide the problem for a winter or two without actually solving it.

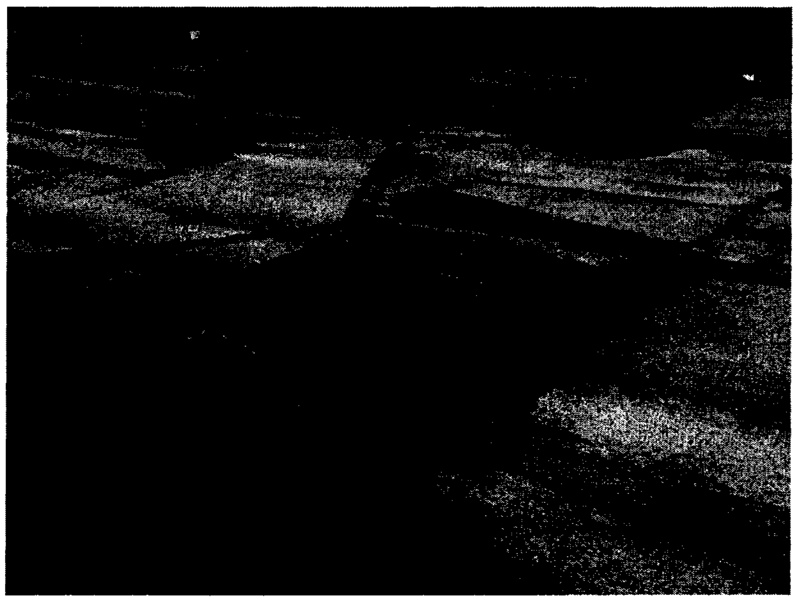

Figure 5. 13: Settled and cracked surface after milling

(Sherbrooke Street, between Redpath and De la Montagne Street, May 2005)

Additionally, manholes and surface drains were not raised prior to the application of the overlay, which resulted in poor compaction around these areas. The deterioration will be similar to what is seen in Figure 5.13 (settlement and cracking) and might exhibit itself shortly after traffic is resumed. The photograph in Figure 5.14, taken on Sherbrooke Street on the 3rd of May 2005, shows the overlaying process around a surface drain and a manhole. The manual compaction effort cannot be expected to achieve the adequate density, as high volume of air voids will be present causing the asphalt to settle significantly under the traffic induced compaction. Furthermore, the asphalt surrounding the surface drain and the manhole will exhibit extensive cracking due to the brittleness of the mix caused by the early age-hardening. In addition to the initial manual compaction, the asphalt overlay is compacted by driving a steel wheeled roller on the final pavement surface, as shown in Figure 5.15. Unfortunately, this method can result in inadequate compaction around manholes, surface drains and unleveled depressions. This is caused by the fact that the steel wheel can easily bridge over the uneven surface, resulting in a laid asphalt mixture with a higher air void content. As discussed previously, the difference in the air void content will result in differential settlement when traffic is resumed, as well as early age-hardening of the asphalt mixture leading to brittle behavior, therefore deteriorating and cracking the pavement surface.

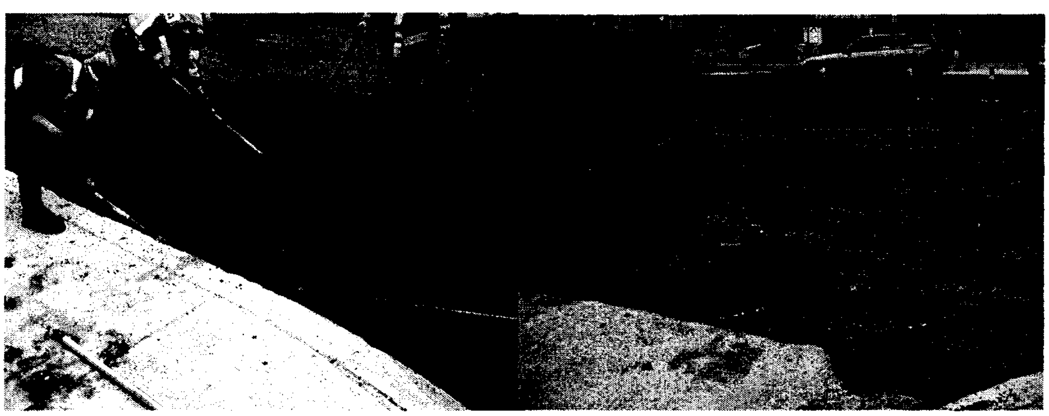

Figure 5. 14: Overlay operations around a surface drain (left) and a manhole (right)

(Sherbrooke Street, between Du Musee and Redpath Street, May 2005)

Figure 5. 15: Compaction with steel wheel roller

(Sherbrooke Street, between Mackay and Crescent Street, May 2005)

According to Mr. Themens, engineer at the Public works office of the City of Montreal, the road infrastructure in Montreal is not adequately documented or detailed as "no two paved sections are identical". In fact, most of the city streets have been constructed in the 1950's without any foundation or subbase as shown in Figure 5.16. The pavement structure initially consisted of a 200 mm (8 inches) concrete slab laid directly on top of the compacted soil or subgrade. The surface was then covered with two, 50 mm thick, asphalt overlays to form the final surface or riding course. These poor construction procedures, where drainage was practically inexistent, may also have contributed to the premature deterioration of the road network. Under traffic loads, the low permeability of the concrete base caused high pore pressures to develop in the undrained and saturated soils from pumping. This resulted in the permanent displacement of materials in the subgrade, leading to the loss of the continuous support of the pavement structure and causing the concrete slab to bridge over the settled sections, therefore overloading the slab causing the failure of the pavement structure and deteriorations to appear at the surface in the form of fatigue cracks and eventually potholes.

In conclusion, the current state of Montreal roads can also be attributed to the lack or deferral of regular, timely maintenance. This may include regular crack treatment and the cleanup of subsurface drains. Additionally, the slow response time (one temporary-patch per nine hours of work) of the city workers to perform selective, low quality emergency repairs can worsen the problem as more surface water seeps in and further weakens the poorly drained subsurface layers. These emergency repair procedures are not sustainable, besides being expensive and inefficient, and they rarely address the real problem which will resurface shortly after the first winter.

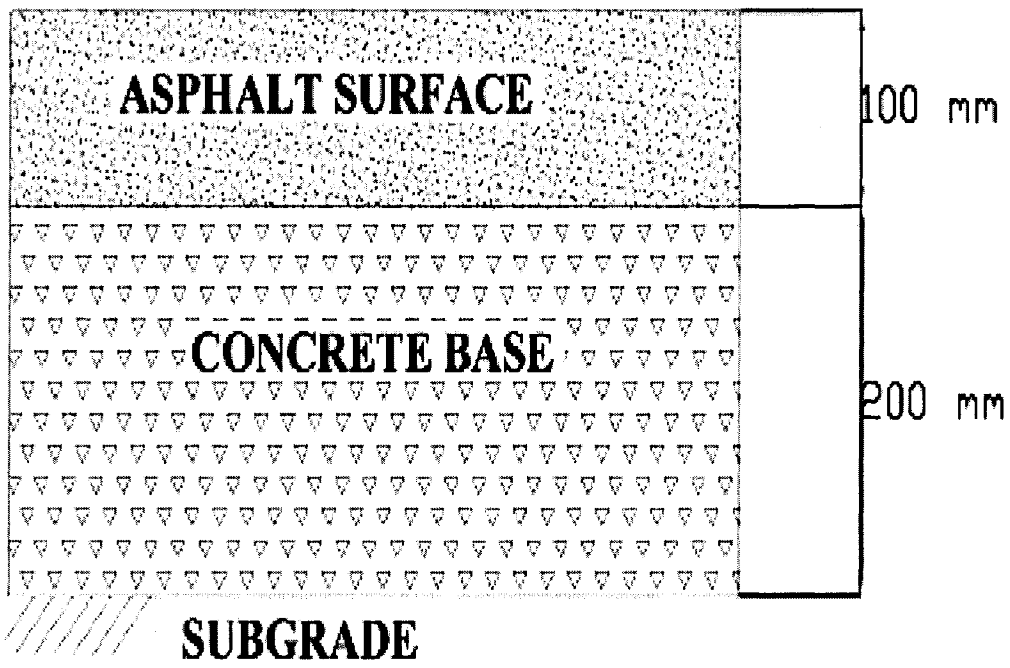

Figure 5. 16: Typical pavement section in Montreal City

(Personal communication with Mr. Themens, Engineer at the City of Montreal)

5.2 Adequacy of Quebec pavement design procedures

Road pavements in the Province of Ontario show less surface deteriorations than those in Quebec. In fact, most Quebecers agree that pavements in the Province of Ontario are at a higher level of serviceability than in Quebec. This can be explained by the difference in spending between the two provinces as shown in Table 5.3, while the differences in construction procedures (Table 5.4) is also a contributing factor in determining the failure or success of the pavement. Table 5.3 shows a significant increase in the total spending in Quebec for pavement systems during the last eight years but it is still $500[1] million per year short of the funding required.

[1] Difference in spending per Km from Table 5.3 multiplied by 29100 km of roads in Quebec

	Quebec Spending 10^6 \$ (29100 km of roads)	Ontario Spending 10^6 \$ (20000 km of roads)	Difference in spending[2] per km 10^3 \$ / km
1996-1997	445	895.7	29.4
2000-2001	705	1121.8	31.9
2003-2004	929.8	1021.5	19.1

Table 5.3: Comparison between Quebec and Ontario Pavement spending

(On Assignment, CFCF News, 2003)

Agency	**Typical Materials Used**	**Minimum Thickness Specifications**
Ontario	Asphalt	40 mm
	Stabilized Drainage Layer	---
	Granular Base	150 mm
	Granular Subbase	---
Quebec	Asphalt	60 mm
	Granular Base	150 mm
	Granular Subbase	300 mm
	Other Subbase	300 mm

Table 5.4: AASHTO flexible design input (C-SHRP, 2002)

[2] The difference in spending per km = (Ontario spending / 20000 km) - (Quebec spending / 29100 km)

Furthermore, Table 5.4 presents the different pavement design inputs used by both Provinces of Quebec and Ontario. A quick examination shows that a stabilized drainage layer is used in Ontario on top of a 150 millimeters thick granular base, which can affect the difference in spending. In both provinces, the edge drains are placed below the frost line as part of the subsurface drainage; unfortunately, in Quebec there is no lateral drainage layer used on top, or below the granular base which is assumed to be free draining. The lateral permeability of a stabilized drainage layer exceeds that of any granular layers which can get clogged due to the rearrangement of the granular particles from sliding and rolling under dynamic traffic loads. The particle rearrangement will impede the drainage capacity of the pavement in both vertical and horizontal directions, resulting in the saturation and weakening of the subsurface layers.

The drainage system of the Montreal roads, if used at all, generally consists of placing longitudinal edge drains 1.85 meters below the level of frost penetration. There are no provisions for channeling the subsurface water towards these drains. Moreover, there is a common trend in reducing the total project costs on the account of drainage. The catastrophic results were observed during the 2005 summer when the newly reconstructed "L'Acadie exchange" was flooded by a 50mm/2-hours rain event. The 110 million dollars project did not include a retrofitting of the drainage system in an attempt to limit the cost. This is unacceptable because of the proven importance of drainage in increasing the service life of the pavement (NCHRP (1997)). Finally, flooding a poorly drained pavement structure will cause water to pond on the surface and the underlying layers to remain saturated for a prolonged period of time. This will weaken the pavement layers and result in unexpected and premature failures when subjected to water expansion upon freezing, under traffic loads or temperature variations. According to NCHRP Synthesis on Highway Practice 96 (1982), drained structures have a service life double that of the undrained ones. The difference in drainage design as well as the difference in yearly spending between the two provinces may be the reason why Ontario pavement structures have proved to be more durable at a higher level of serviceability than in Quebec.

Another aspect of this analysis is the design procedure used in Quebec. As discussed in Chapter Two, the 1986 AASHTO design procedure is purely empirical and results in pavement structures that cannot be expected to perform adequately under extreme loading conditions. Furthermore, the AASHTO design equation was based on an 80kN ESAL (Equivalent Single Axle Loads) with a maximum allowable axle load of 142 kN (32 kips) (see Figure 5.18), whereas the maximum allowable axle load according to the CHBDC (Canadian Highway Bridge Design Code) is 175 kN (40 kips) (Figure 5.17). Additionally, the actual loads that act on the surface of the pavement are always higher than the static design loads due to their dynamic/ bouncing nature (impact). The actual value of the loads depends on the roughness of the pavement, the speed of the traffic, and the type of vehicles, tire pressure and suspension used and is accounted for in the design by introducing a dynamic load allowance factor. The 25% increase in the applied design loads (175kN /142kN) combined with higher legal loads and different dynamic load allowance in Canada compared to the U.S.A results in the application of heavier traffic loads which leaves our pavement design short on thickness.

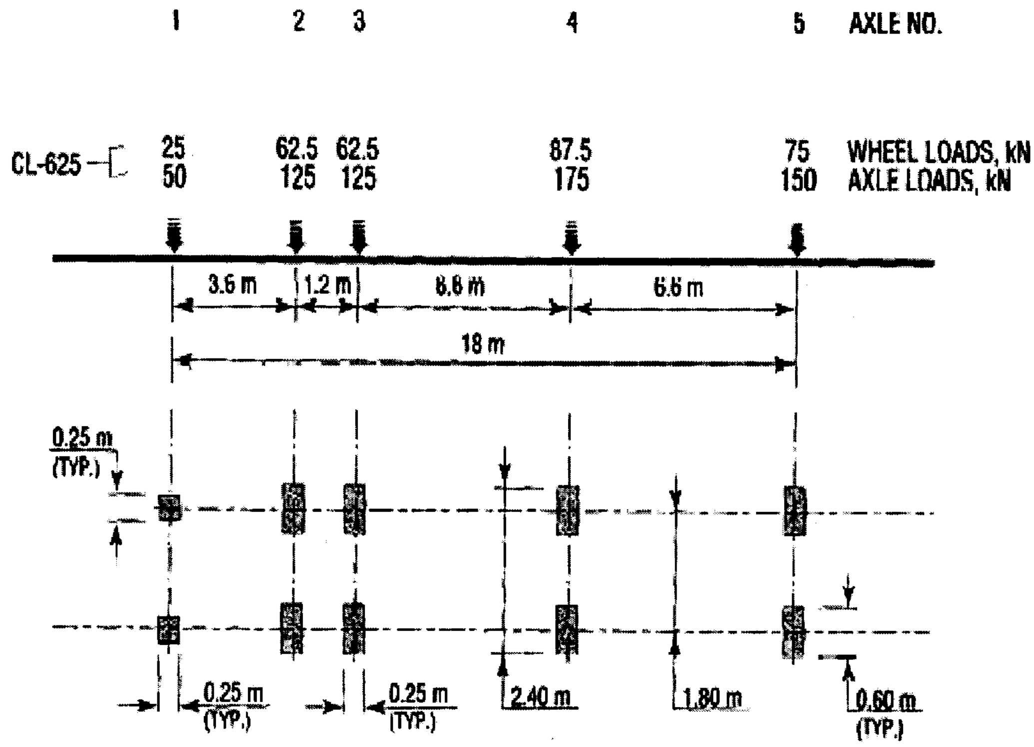

Figure 5. 17: CL-625 CHBDC truck load (Kennedy et al., 2002)

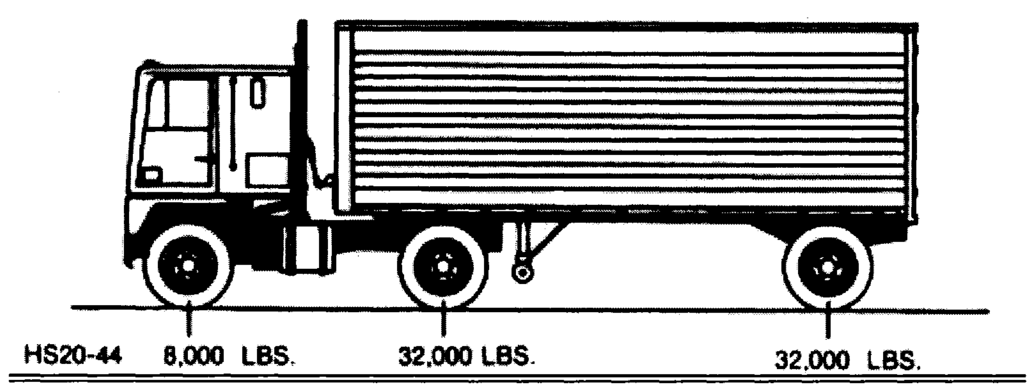

Figure 5. 18: HS20 AASHTO truck load (Nystrom et al., 2002)

It can be concluded that road pavements in Quebec and more specifically in Montreal are poorly drained and subjected to loads higher than the initial design loads. Additionally, the high (25%) increase in daily traffic volumes on the Island of Montreal during the last decade (Quebec Ministry of Transportation, 2000), as well the global climatic changes, such as global warming, have contributed to the worsening of the problem. The premature failure of the pavements in Montreal can be attributed to the weakened subsurface layers due to poor drainage (loss of stiffness) and extended saturation times, as well as to the weak pavement design procedures which result in insufficient thickness of the pavement. These observations can explain the extensive pothole and fatigue cracking failures which are caused by the weakening of the pavement structure.

5.3 Proposed intervention procedures:

According to Mr. Claude Dauphin (member of the City executive committee), around 60% of Montreal city's main arterial roads have been in service for more than 50 years and need major rehabilitation, upgrading, or even reconstruction. This is a major turning point which will require accurate planning, design, construction and funding to achieve a high level of serviceability and durability. These are keys to successful local and national economy. The City has recently announced the creation of a "road network fund" to ensure long-term financing for maintenance and rehabilitation of the road pavement with a budget of $500 million by 2009. Unfortunately, this money can be wasted without improving the serviceability and condition of the road pavement if the current repair and construction strategies are maintained.

There is an urgent need to develop performance-based designs that can accurately predict and model the response and performance of the pavement under actual dynamic traffic loads. Moreover, there is a strong need to improve the surface and subsurface drainage systems as water cannot be expected to reach the drains without adequate channeling. Surface water can be channeled by designing flow paths within the macro-texture or by sawing shallow grooves in the surface of the asphalt layer, while free water within the pavement layers must be drained laterally towards the edges for fear of ingression towards the lower layers. Additionally, moisture rise within the pavement caused by the capillary suction of the soil cannot be drained and must be stopped to inhibit the formation of ice lenses. This can be achieved by placing a geocomposite membrane that can act as a barrier against these moisture movements, below the subbase and these can be connected to the edge drains.

Finally, the lack of investment as well as the current "build and forget" practice, resulting in deferred maintenance that has been in effect during the last few decades has increased the repair costs and the deterioration rate of the pavement as shown previously in Figure 4.1. Figure 5.19 presents a qualitative comparison between the effects of regular preventive maintenance versus the "build and forget" strategy followed by a major rehabilitation, on the serviceability and durability of the pavement structure. The results show that preventive maintenance plans can effectively restore the initial condition of the pavement, while reducing its deterioration rate resulting in a more durable, less costly, high performance pavement. The dotted curve, which corresponds to the case of deferred maintenance in Figure 5.19, shows an increasing deterioration rate with time leading to a shorter service life and requiring costly rehabilitation.

Preventive pavement maintenance has proven to be efficient in increasing the performance and durability of the pavement with reduced spending over the life of the structure. It must be seriously considered when planning the rehabilitation, or even new construction of a pavement structure.

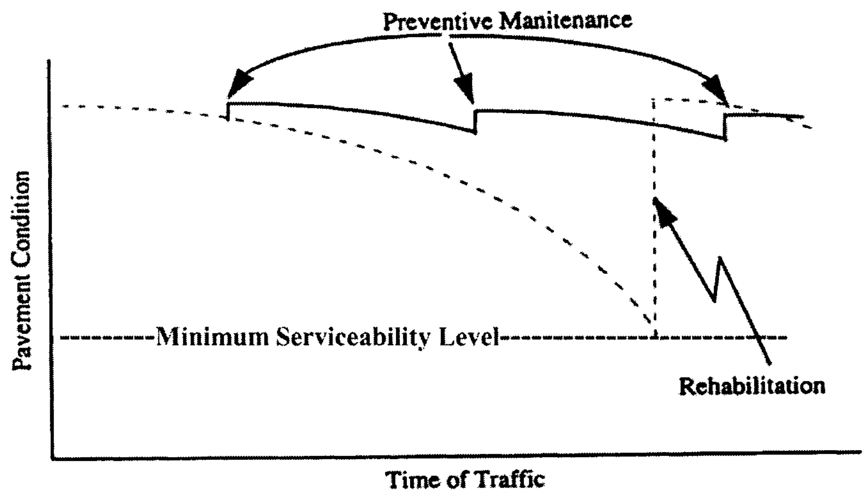

Figure 5. 19: Effect of timely preventive maintenance on pavement performance (Mamlouk and Zaniewski, 1998).

The implementation of a preventive maintenance and timely repair plan must be directed by a politically independent committee so that it is not subjected to deferrals, short-term vision or budget cuts.

Chapter Six

Conclusions

6.1 Summary and Conclusions:

This research program was aimed at determining the main problems that affect the asphalt pavements in Montreal and lead to their premature failure. The different observations that were recorded were assessed in terms of drainage, materials and adequacy of design and construction procedures. The following conclusions can be drawn:

1. The deferred maintenance caused by budget cuts, lack of future vision in implementing adequate rehabilitation and reconstruction strategies, as well as the slow repair/intervention time of the City workers and officials are a major factor in determining the poor performance and durability of the pavement. Their importance has been established and proven since the deterioration rates increase exponentially if the deterioration problems are not handled in time and only local emergency repairs are randomly performed; this is contrary to the principles of sustainable development.

2. The use of the 1986 AASHTO empirical pavement design procedures in Quebec has proven to be inadequate as empirical relationships cannot accurately predict the pavement performance. Furthermore, this study has also determined that the maximum allowable axle load and the maximum truck load are higher in Canada than in the United States, where these design

equations were developed. The pavement is therefore subjected to loads higher than those used in the initial design, which explains the premature fatigue failures

3. Transverse drainage was defective as no transverse drainage layer is used and no effort is provided to channel the free water towards the drains. This allows the water to collect and pond within the low permeability pavement layers causing loss of stiffness, material displacement and loss of continuous support leading to permanent deformation and premature failure of the pavement.

4. The lack of documentation, detailing and standardization of the city streets have resulted in random repairs, rehabilitation and reconstruction strategies. Additionally, the old and inadequate construction practices, consisting of a low permeability concrete slab laid directly on top of the existing and undrained soil, caused high pore pressures to develop in the saturated subgrade under heavy traffic loads, due to pumping. These high pressures caused permanent material displacements below the slab leading to the loss of continuous support thus the formation of fatigue cracks initially then potholes.

5. The study has also determined that potholes, fatigue/alligator cracking as well as failed patches are the most frequent deterioration modes that were observed around the City of Montreal. This was expected since they all involve the presence of water in a weak pavement structure. Moreover, inadequate repair planning and techniques that were observed and previously discussed, as well

as poor quality control have also contributed to the wastage of the available funds, and postponing the problem to the next winter.

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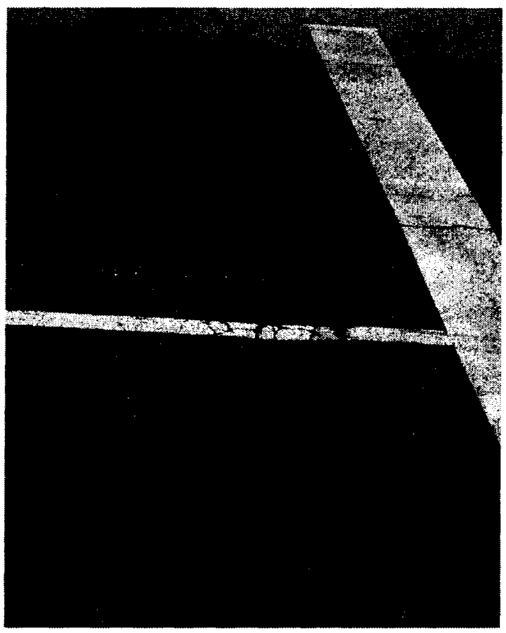
Appendix A

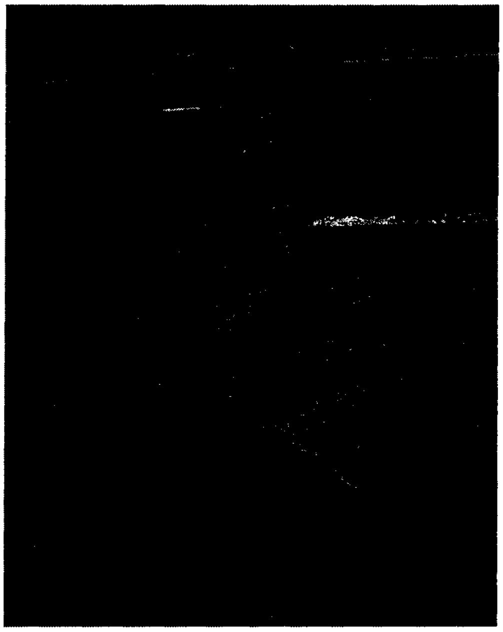
The complete pictures are presented on a compact disk with Professor Saeed Mirza of McGill University, and they are grouped by the streets.

A.1 Thermal cracks

A. 1.1 Du Parc Ave., corner of Duluth St.

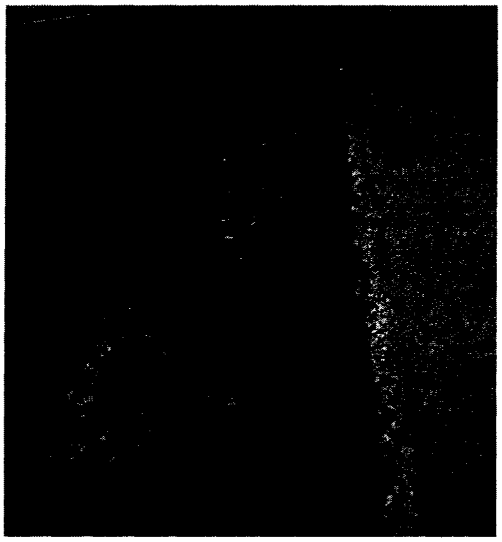


A.1. 2 Rene Lévesque St., corner of Du Fort St.

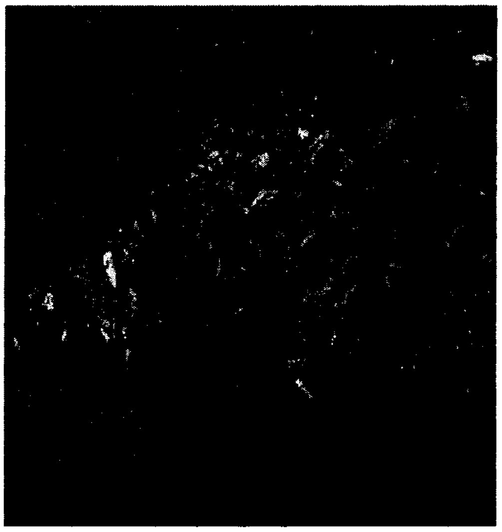


A.2 Wear loss

A. 2.1 Cote Ste-Catherine, corner of McNider Ave

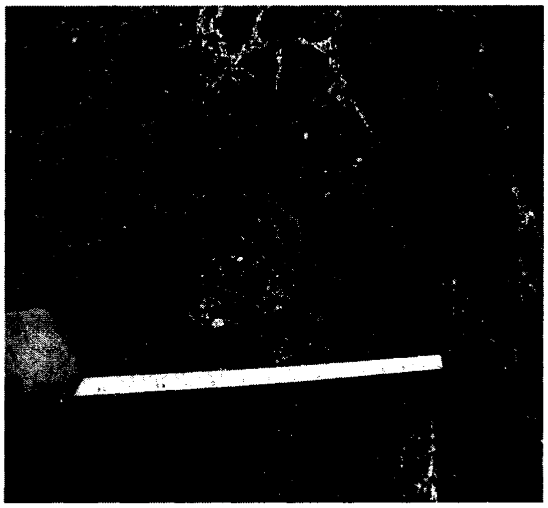


A.2. 3 Cote Ste-Catherine, corner of Mt-Royal

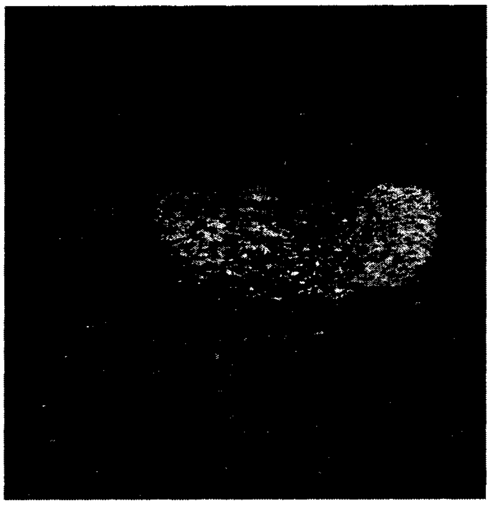


A.3 Potholes

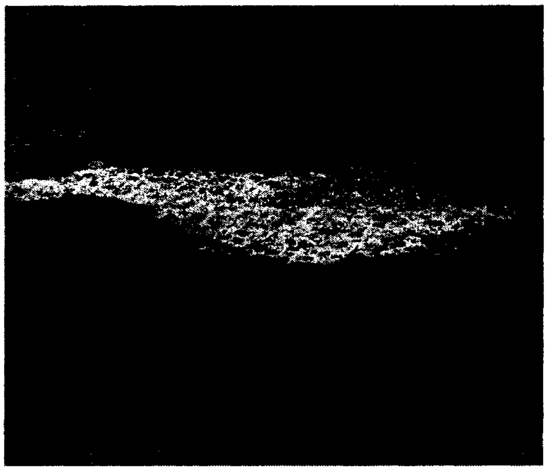
A. 3.1 Cotes des Neiges, corner of Gage dr.



A. 3. 4 Des Pins Ave., corner of Aylmer St.

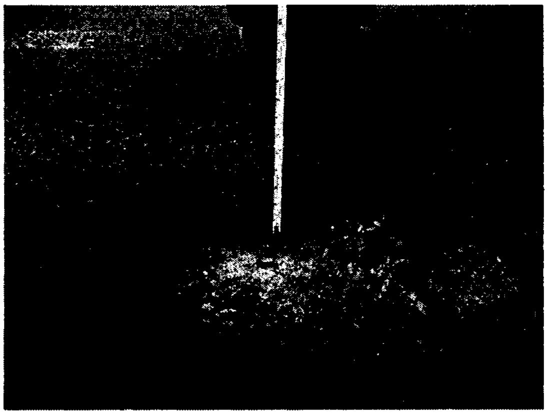


A. 3. 3 Des Pins Ave., corner of Durocher St.

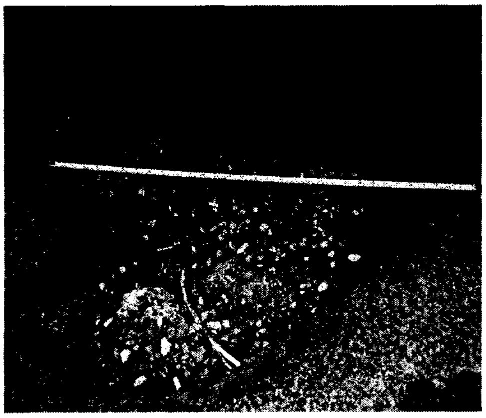


A. 3. 4 Du Parc Ave., corner of Des Pins Ave.

A. 3. 5 Du Fort St., corner St-Catherine St.

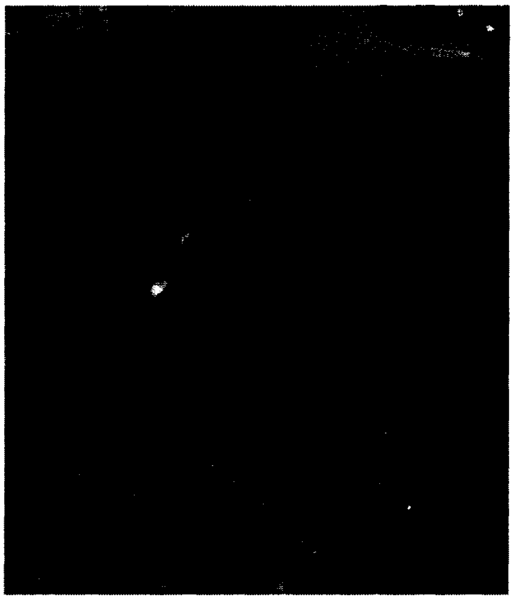


A. 3. 6 Du Fort St., corner Rene Lévesque St.

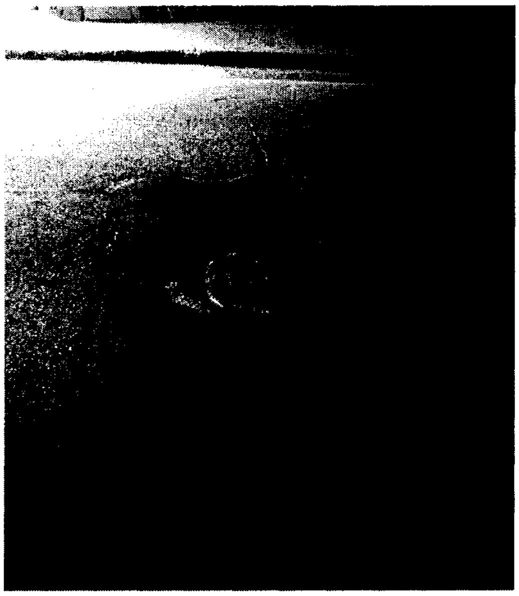


A.4 Permanent patches

A. 4. 1 De Maisonneuve Blvd, corner Guy St.

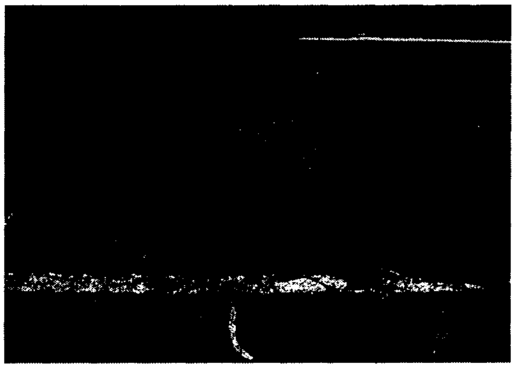


A. 4. 2 Des Pins Ave., corner of Du Parc Ave.



A. 4. 3 St-Catherine St., corner of Bishop St.

A. 4. 4 Du Parc Ave., corner of Duluth St.



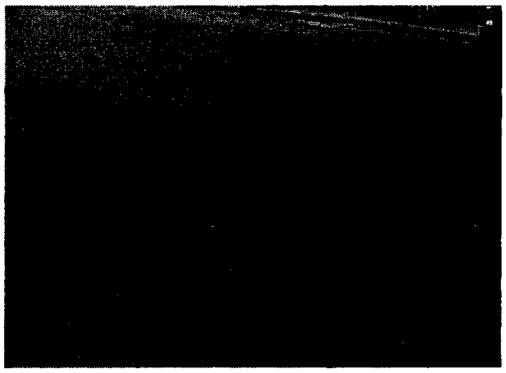
A. 4. 5 Des Pins Ave., corner of Durocher St.

A. 4. 6 St-Catherine St., corner of Guy St.

A.5 Temporary patches

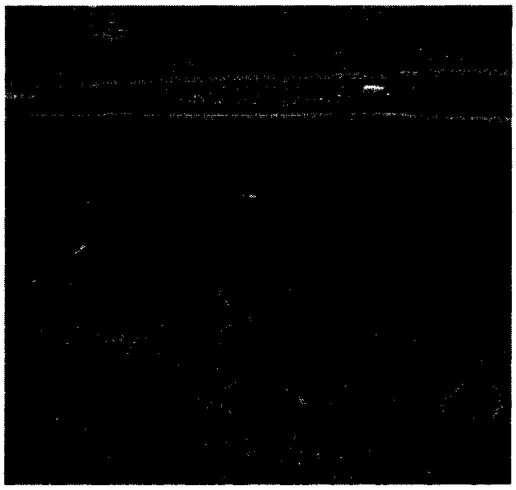
A. 5. 1 Du Fort St., corner of St-Catherine St.

A. 5. 2 St-Catherine St., corner of St-Mathieu St.

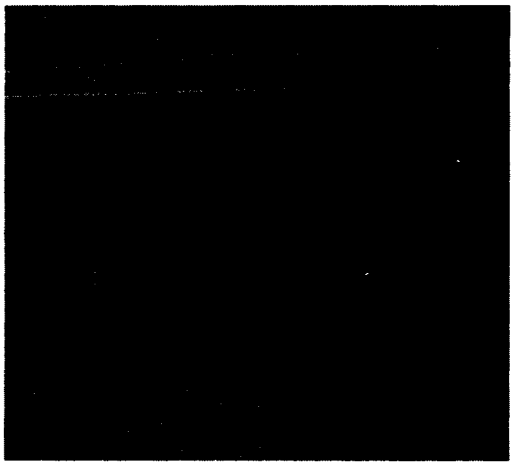


A. 5. 3 Des Pins Ave., corner of Ste-Famille St.

A. 5. 4 Des Pins Ave., corner of Du Parc Ave.



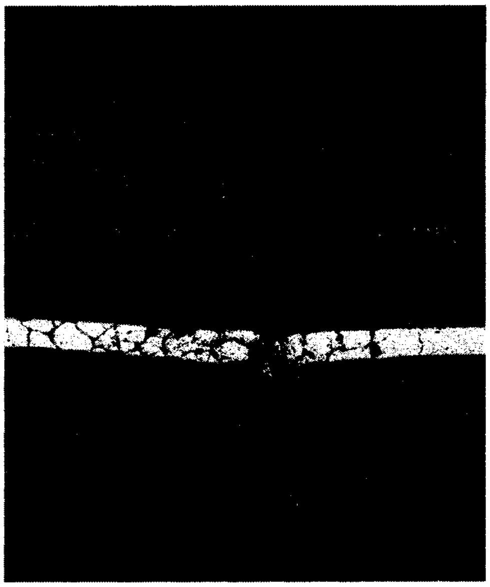
A. 5. 5 Des Pins Ave., corner of Du Parc Ave.



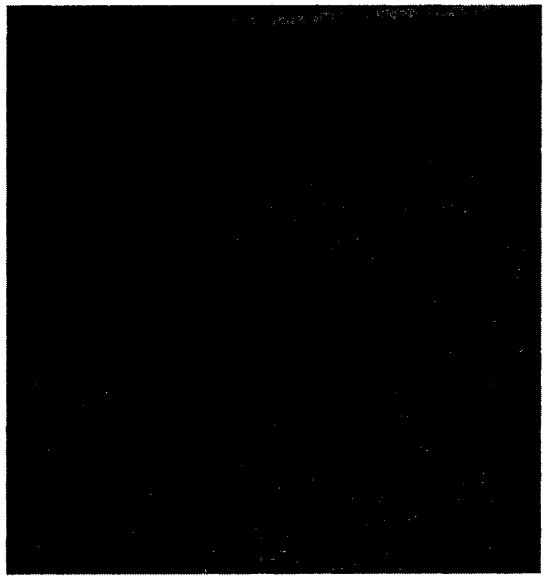
A. 5. 6 St-Catherine St., corner of St-Marc St.

A.6 Fatigue cracks

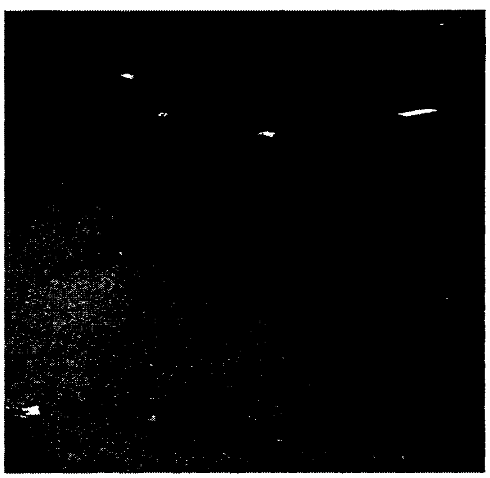
A. 6. 1 Du Parc Ave., corner of Duluth St.



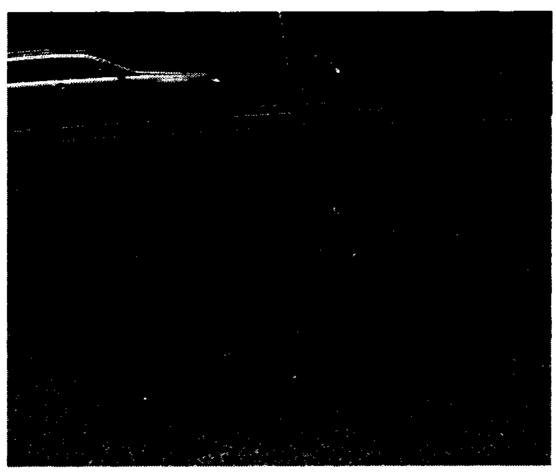
A. 6. 2 Cotes des Neiges, corner of Decelles Ave.



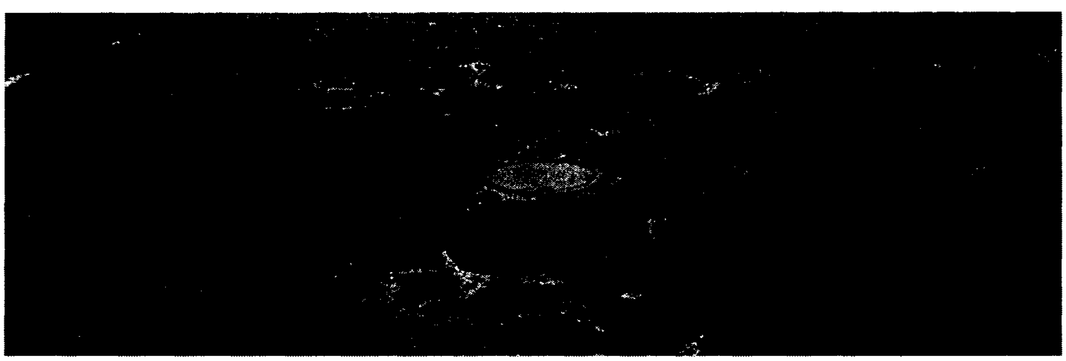
A. 6. 3 Pierce St., corner of De Maisonneuve



A. 6. 4 St-Catherine St., corner of Pierce St.



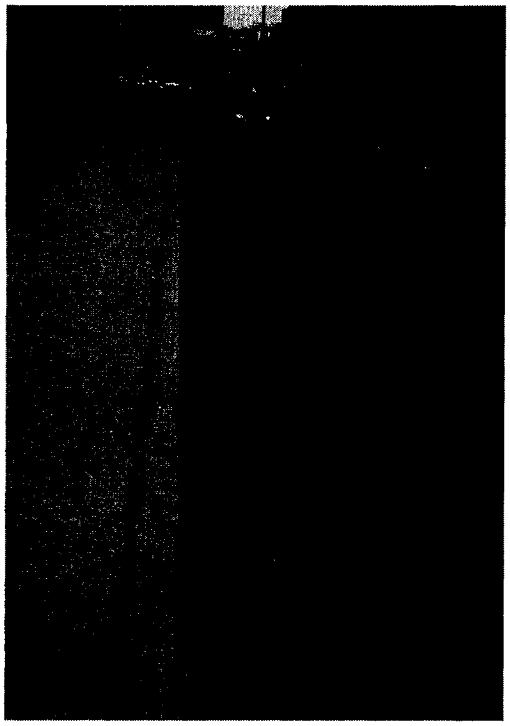
A. 6. 5 Cote Ste-Catherine, corner of Mt-Royal St.



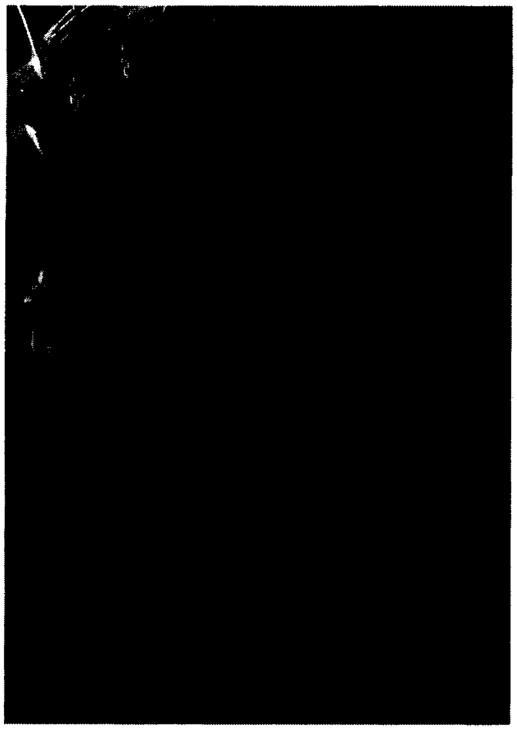
A.7 Longitudinal cracks

A. 7. 1 St-Catherine St., corner of Du Fort St.

A. 7. 2 Des Pins Ave. facing McGill Gymnasium



A. 7. 3 St-Catherine St., corner of Guy St.



A. 7. 4 Des Pins Ave. facing McGill Gymnasium