# LONG TERM PAVEMENT PERFORMANCE OF THE JAMES BAY ACCESS ROAD

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## ABSTRACT

The James Bay Access road was constructed in 1972-74 to provide access to the James Bay Hydro Electric project on the La Grande River. The 620 km access road over difficult terrain conditions was constructed in just 450 days. This modern highway was paved in 1975 and 1976.

A major concern was thermal cracking in this harsh climatic environment and as a consequence the road was one of the first major highways in Canada to be paved using a 300-400 penetration grade asphalt.
The road has been subjected to intensive truck traffic as it provided access for all cement, fuel and construction materials required for the hydroelectric project. In addition, there were a number of special hauls required to transport 500 tonne transformers.

This paper reviews the pavement condition after 0,1,2,3,5,10, 20 and 23 years. This data provides a unique large scale study of the performance of a pavement subjected to severe traffic and climatic conditions. An analysis of the data indicated that pavement performance, particularly ride score, wheel track cracking and rutting were strongly dependent on subgrade soil type. Thermal cracking frequency was significantly higher on granular deposits and on sections that had intensive hauling related to dam construction.

## RÉSUMÉ

La route de la Baie James a été construite en 1972-1974 afin de donner accès aux aménagements projetés pour la complexe hydroélectrique La Grande. Cette route d’accès de 620 km, traversant un territoire où les conditions physiographiques étaient difficiles, a été complétée en 450 jours. La route a été pavée en 1975 et 1976.

À cause des conditions climatiques rigoureuses, la résistance à la fissuration par temps froid du revêtement bitumineux était une préoccupation majeure des concepteurs. Par conséquent, la route de la Baie James fut l’une des premières routes d’importance au Canada à être pavée en utilisant un ciment asphaltique ayant un indice de pénétration 300-400. Cette route a été soumise à un intense trafic lourd puisqu’elle a servi à transporter le ciment, les carburants et tous les autres matériaux nécessaires à la construction des aménagements hydroélectriques du territoire. Elle a également dû supporter un certain nombre de chargements spéciaux pour le transport de pièces extra-lourdes allant jusqu’à 500 tonnes, comme les transformateurs.

Cet article décrit l’évolution des conditions du revêtement bitumineux après 1,2,3,5,10,20 et 23 ans. Ces données constituent une occasion unique d’évaluer la performance d’un pavage soumis à des conditions climatiques et de trafic extrêmes et ce, sur une aussi grande échelle. Une analyse de données a permis de déterminer que la performance du revêtement dépend fortement du type de sol sous-jacent, particulièrement en ce qui a trait à l’uni de la chaussée ainsi que l’orniérage et la fissuration au droit du passage des roues des véhicules. La fréquence des fissures thermiques est supérieure de façon significative au-dessus des dépôts granulaires ainsi que dans les sections sujettes à la circulation intense reliée à la construction des barrages.

## 1.0 INTRODUCTION

This paper deals with the performance of 620 kilometres of pavement on the James Bay Access Road. (Figure 1). The road was paved with 300/400 penetration grade cement in 1975 and 1976 and marked one of the first major projects where this grade of asphalt cement was used. This paper reviews the pavement condition after 0,1,2,3,5/6,10/11, 20/21 and 23/24 years. The data provides a unique large scale study of the performance of a pavement subjected to severe traffic and climatic conditions. Approximately 50% of the original pavement is still in service. This paper deals mainly with the performance of the highway between km 0 and km 582, which was subjected to Aregular@ highway traffic. Unless specifically mentioned, the sections between km 582 and 620 are excluded from the study, as these sections were subjected to intensive hauling of sand and gravel for dam and dyke construction.

This paper is divided into 2 main investigations. The first part deals with 10 test sections, each 1.6 kilometres long, where cracking patterns were mapped after 1,2,5 and 24 years of service. As part of this study, samples were taken at two of these test sites in 1999 and the asphalt properties including the SHRP low temperature Performance Grade established. The second part of the study reviews the performance of the roadway from a pavement management perspective. Some 30 sections, each 20 km long have been rated at various times over the last 24 years and the overall performance of the sections is evaluated. In addition to providing guidance for the rehabilitation of the highway, this study provides information for future paving in the area. Recently roads have been or are being constructed into the James Bay Settlements of Wemindji, Eastmain, Waskaganish, Route du Nord et la route Nemiscau and represent a potential of 700 kilometres of new pavement in the area.

The pavement management part of the study classified the pavement subgrades into five major soil types ranging from very weak muskeg-clay complexes to massive sand and gravel deposits. The behavior of pavements on these soils is studied based on this classification.

## 2.0 BACKGROUND

The access road to the James Bay Hydroelectric site rates as one the most ambitious highway projects ever undertaken, paralleled only by the construction of the Alaska Highway during the second World War. The James Bay Access Road including the spur road to James Bay at Chisasibi is some 720 kilometres in length and was built at cost of $225 million in 1976 dollars.

Construction started in the winter of 1971/72 as contractors mobilized to construct an access from roads- end at Matagami to the Rupert River some 260 kilometres away. The purpose of the road was to provide access to the NBR (Nottaway, Broadback and Rupert Rivers) hydroelectric project. The contractors mobilized fuel, equipment and camps over a winter road as they would be isolated until November of 1972 when the Acarriageable roadway@ was to be completed. By definition a Acarriageable roadway@ was one Athat permitted access by a loaded float truck (a truck for transporting construction equipment) at 15 mph (25 kph).@ The carriageable roadway was completed in late October 1972 and provided access to Rupert River. This section was completed with a great deal of difficulty as the winter road was constructed late in the season and barely existed in some sections. The terrain was difficult as much of the road traversed very soft sensitive marine clay deposits. Many of these deposits were covered with one to two metres of muskeg.

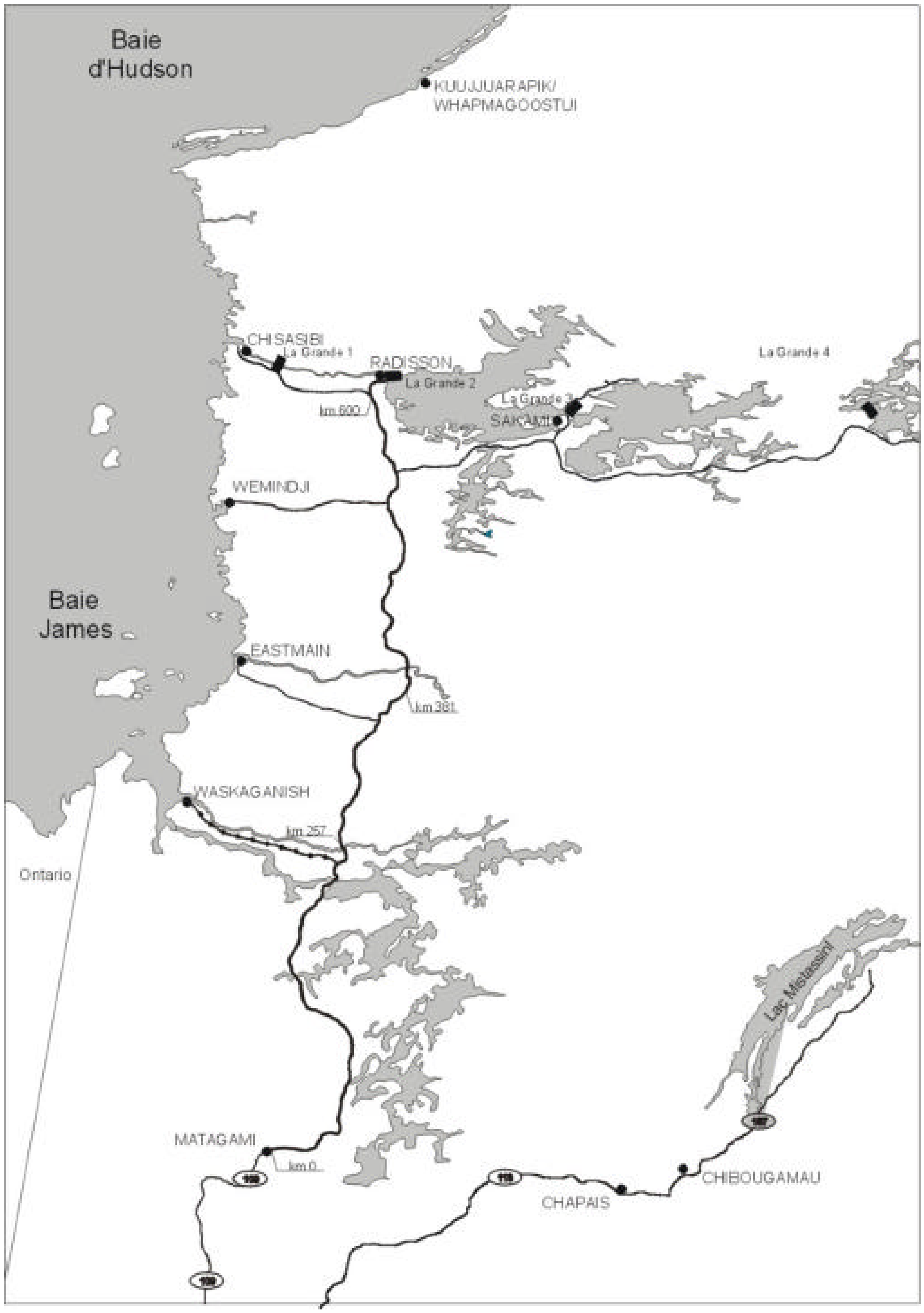


**FIGURE 1: LOCATION MAP**

While some attempts were made to remove the muskeg, most of the road was Afloated@ over the muskeg without removing it. In areas where the muskeg was removed, the underlying clay material was in most cases, a poorer foundation material than the original muskeg. In many areas, suitable subgrade construction material was scarce. In one instance, a fine sand pit provided the only material for construction of 65 kilometres of the road. Despite all the obstacles, the road was Aroughed in@ to meet the tight schedule. For the most part, the subgrade was constructed to the full width during this phase to minimize differential settlements.

In the spring and summer of 1973, the subgrade of the section from km 0 to km 298 was completed using normal construction practices and associated quality control. The highway had a road top width of 13.4 metres and a design speed of 100 kph. The roadway was surfaced with a crushed base course obtained mainly from quarries.

In the summer of 1972, Hydro Quebec changed the project substantially when it decided to proceed with the La Grande phase of the project some 300 kilometres further north instead of the NBR project. Initially a 90 kilometre a temporary low standard road was constructed from Fort George (near present day Chisasibi) on James Bay to the future site of the main dam LG2 in the summer of 1972. Two contractors then mobilized to construct a permanent road from Fort Georges to LG2 by mobilizing from Montreal through Hudson Strait to Fort George and the temporary road.

Contractors for the 300 kilometres from the end of the 1972 construction to LG2 mobilized in the winter of 1973 using a winter road. The terrain on this phase of the work was more favorable. It consisted of rock, muskeg and till materials. Relatively shallow sand and gravel deposits were used for subgrade, subbase and base materials. There were also two very large sand and gravel deposits near the north end of the road that facilitated construction in those areas. A Acarriageable roadway@ between km 298 and km 620 - the dam site, was completed by the end of October 1973. Four hundred and fifty miles of access road had been completed just 450 days after the first survey crews arrived on the project.

The section from km 298 to km 620 (LG2) was completed as a gravel surfaced highway on October 20, 1974. No longer a trail in the bush, the road was complete and was the first road in Canada to be signed completely using the SI system.

In the summer of 1974, a Life Cycle Cost study indicated paving the access road was the most favorable economic alternative. Hydro Quebec was in a unique position, for not only was it responsible for the construction and maintenance of the roadway, but as the sole user, any savings in transportation of materials were direct savings to the company. Many agencies use construction and maintenance costs for comparing alternatives but since user costs are Aindirect,@ there is always some discussion on the inclusion and/or determination of user costs. In this case, the user cost savings were real and direct to the owner.

Based on a 10 year hauling span, the estimated user and maintenance costs were $120 million for the gravel alternative. Paving costs ($30 million), user costs and maintenance costs for a paved surface were estimated at $76 million for the same 10-year span. Consequently based on the projected savings of $44 million, the decision to pave was made.

The road was paved in 1975 and 1976. The sections from km 0 to km 185, from km 240 to km 298 and from km 547 to km 620 were selected for paving in 1975. The rational for paving these sections was that the sections from 0 to 298 had been completed in 1973 and the subgrade had two years for settlement to occur. The section between km 185 and km 240 had been built over very difficult terrain and it was felt desirable to allow an additional year of settlement. The rational for paving km 580 to km 620 near the dam site was due to the logistics of keeping a gravel road serviceable for more than 2,000 trucks a day. Despite three graders working 24 hours a day and a calcium application every two weeks, the road was barely maintainable and there were safety concerns as well as productivity problems.

Ten paving contracts of approximately 60 kilometres each were called - five in 1975 and five in 1976. Four of the five contracts in 1975 used quarries for aggregate sources. As there was some concern about stripping, a chemical anti-stripping agent was used in the 1975 contracts.

The logistics of supplying five contractors 1500 kilometres from the refinery resulted in the installation of a transhipment plant in Matagami. Asphalt was shipped by rail from Montreal to Matagami where it was reheated before being pumped into highway transports for delivery to the job sites. Orders were placed with the supplier for next day delivery. The supplier was required to have a two-day supply of asphalt at all times in Matagami.

## 3.0 MATERIALS

### 3.1 Asphalt Cement

#### 3.1.1 Penetration Grade

The rational for the selection of the asphalt cement has been well documented [1]. To provide complete protection against low temperature transverse cracking at a design temperature of - 51.1$^0$C would have resulted in an asphalt cement with a penetration of a 1000 at 25$^0$C. This would be approximated by SC3000 or SC5 of low temperature susceptibility. However, as there was some concern due the intensive truck traffic, it was decided to select an asphalt cement of 300/400 penetration having a low temperature susceptibility (minimum PVN + -0.5). This would hopefully provide adequate pavement stability for warm weather traffic although it would result in some low temperature cracking, since the design was for -40 $^0$C rather than to the -51.1$^0$C to which it would be exposed [1]. Table 1 lists testing of the original asphalt and recovered asphalt as reported by Lefebvre [1] and Hode Keyser [2].

#### 3.1.2 Performance Grade Requirements

With the advent of Performance Grade binders, it is of interest to note the grading required for the pavements if they would have been constructed with PG grade asphalts. Weather records are available for Matagami and La Grande (it should be noted that there were no weather records for La Grande until 1977 and the models would have been of limited use). The PG requirements have been calculated using two procedures - the FHWA-LTPP model [3] and Robertson's revised equation [4].

FHWA-LTPP equation:

$$T_{pav} = -1.56+0.72\ T_{air} - 0.004\ Lat^2 + 6.26\ \log_{10}(H+25) - z\ (4.4+0.52(sd_{air})^2)^{0.5}$$

where $T_{pav}$ = Low asphalt concrete temperature below surface $^0$C
$T_{air}$ = Low air temperature $^0$C
Lat = Latitude of Pavement - degrees
H = Depth below surface mm
$sd_{air}$ = Standard deviation of the mean low air temperature $^0$C
z = Multiplier associated with desired reliability

Robertson equation: $T_{pav} = 0.749\ T_{air} - z\,(0.749(sd_{air})^2 + sd^2_p)^{0.5}$

where $T_{pav}$ = Low asphalt concrete temperature below surface $^0$C
$T_{air}$ = Low air temperature $^0$C
$sd_{air}$ = Standard deviation of the mean low air temperature $^0$C
$sd_p$ = Standard error of the estimate of pavement surface temperature - 1.5 $^0$C
$z$ = Multiplier associated with desired reliability

Table 2 indicates the PG Grades required for Matagami and La Grande.

**Table 1 Inspection Data - 300/400 Penetration Asphalt Cement - James Bay Access Road**

| Item | Number of Tests | Mean | Standard Deviation |
|---|---|---|---|
| **Original Asphalt [1]** | | | |
| Penetration at 25$^0$C | 18 | 317.6 | 23.04 |
| Kinematic Viscosity at 135$^0$C cs | 19 | 163.5 | 4.91 |
| Pen-Vis Number (PVN) | 3 | -0.17 | 0.057 |
| Ductility at 15$^0$C cm | 18 | 133.1 | 17.923 |
| **Recovered Asphalt ( Abson) [1]** | | | |
| Penetration at 25$^0$C | 4 | 171.5 | 7.81 |
| Kinematic Viscosity at 135$^0$C cs | 4 | 203.5 | 6.1 |
| Absolute Viscosity at 60$^0$C poises | 4 | 545.8 | 29.7 |
| Pen-Vis Number (PVN) | 4 | -0.65 | 0.022 |
| **Recovered Asphalt after 3 years [2]** | | | |
| Penetration at 25$^0$C | 6 | 145.16 | 20.42 |
| Temperature Ring and Ball $^0$C | 6 | 43 | 0.83 |

**Table 2 : Performance Graded (PG) Asphalt Cement Requirements**

| LOCATION | METHOD | 50% Reliability | 98% Reliability |
|---|---|---|---|
| MATAGAMI | FHWA-LTPP | PG 46-34 | PG 52-40 |
| | Robertson | PG 46-34 | PG 52-40 |
| LA GRANDE | FHWA-LTPP | PG 46-34 | PG 52-40 |
| | Robertson | PG 46-34 | PG 52-34 |

### 3.2 Asphalt Concrete

The asphalt concrete used was a dense graded mix with a 19-mm top size. The mix was required to have 50% manufactured fines to increase stability and enhance resistance to rutting. A large number of tests were run to check gradations, asphalt contents, densities and thicknesses. The Marshall Stabilities were all above the minimum 4,500 N required. The VMA exceeded the 14% required and the air voids were well within the range of 2 to 5 %. Average compaction exceeded the 97% required. [1]**.** A summary of the asphalt concrete mix properties is found in Table 3.

**Table 3: Summary of Quality Control Properties of the Asphalt Concrete**

| **Property** | **Mean** |
|---|---|
| Marshall Stability - N | 6775 |
| Marshall Flow 0.01 inch | 2.45 |
| Voids in Mineral Aggregate (VMA) % | 15.37 |
| Air Voids % | 3.22 |
| Asphalt Content % | 5.49 |
| Compaction % of Lab Density | 98.2 |

The asphalt concrete was laid as a single lift 62.5 mm in depth and 7.3 m wide with the exception of km 582 to km 620 where the total thickness was 100 mm and the width was 9.1 m. The base course was not primed.

### 3.3 Base and Subbase

The base course consisted of two different materials. The top 150 mm of base course was a 19-mm maximum size crushed aggregate material. This was underlain by 300 mm of a crushed material - maximum size 75 mm . The base course was quarried material for the section from km 60 to 298 and was crushed gravel in all other sections. Subbase thickness varied from 0 to 600 mm depending on soil type. Subbase material ranged from fine sand to pit-run gravel. The only requirement for subbase material was that it have less than 10% passing the 0.075 mm sieve.

### 3.4 Subgrade Soil Types

The 620 kilometres of road traversed a wide assortment of soil types ranging from soft sensitive clays, at times covered with deep muskegs to large deltaic sand and gravel deposits. For purposes of this paper, the various sections have been sorted into 5 distinct classifications based on the terrain type. Although very few of the sections were homogenous as many contained varying amounts of the other soil types, the overall classification is typical of the classification that would be used for a conceptual pavement design. Table 4 contains the sections as per the classification used in this study.

**Table 4 : Classification of the James Bay Highway by Subgrade Soil Type**

| SOIL | LOCATION KILOMETRE | COMMENTS |
|---|---|---|
| 1 | 60-80 | Soft sensitive clays - muskeg removed and backfilled |
| | 185-200 | 1-2 metres of muskeg over very soft sensitive clays |
| | 544-560 | Discontinuous Permafrost |
| | 610-620 | Discontinuous Permafrost |
| 2 | 80-185 | Approx. 1 metre of muskeg over soft sensitive clay |
| | 298-320 | Silty till deposits |
| | 380-420 | Deep sensitive clay deposits |
| 3 | 0-60 | Silty till deposits |
| | 240-298 | Silty till deposits |
| | 600-610 | Bouldery tills, shallow rock cuts and fills |
| 4 | 320-380 | Bouldery tills, shallow rock cuts and fills |
| | 420-460 | Bouldery tills, shallow rock cuts and fills |
| | 520-544 | Bouldery tills, shallow rock cuts and fills |
| | 574-582 | Bouldery tills, shallow rock cuts and fills |
| 5 | 460-520 | Massive sand and gravel deposits |
| | 582-600 | Massive sand and gravel deposits |

**Soil 1 :**
This classification was used for the weakest subgrade soils. These soils are classified as very soft sensitive marine clays typically overlaid by one to two metres of muskeg. The sand subbase was generally 1.2 metres thick and was required as a construction expedient as this depth was required to support loaded construction trucks. The California Bearing Ratio (CBR) was assumed to be 1.5 to 2 for design purposes. Soil 1 also contained sections where pockets of permafrost were found.

**Soil 2 :**
Subgrade soils in this classification were also soft marine clays but generally with less than a metre of muskeg on the surface. The design subbase depth was 0.6 m but in many cases a metre of granular material was required for construction expediency. The design CBR was 3 for these materials.

**Soil 3 :**
These subgrades consisted mainly of silty tills at high moisture contents. Although competent in cut sections, the material was rarely used for fill material. These sections had some shallow rock cuts and thin muskegs. The subbase thickness was 0.6 metres. The design CBR was 5.

**Soil 4 :**
These subgrades consisted of more bouldery tills with less silt content than Soil Type 3. These sections had significant shallow rock cuts and fills. Subbase thickness was 0.6 metres. The assumed CBR of this soil type was 8.

**Soil 5** :
These subgrades consisted of deep sand and gravel deposits. Subbase was not required on these sections. The design CBR was 15.

The CBR values listed above were based on a combination of CBR soaked tests, vane shear tests, plate test values and Benkelman Beam deflections. The structural design of the pavement was guided by Asphalt Institute and AASHTO thickness design criteria [1]. Generally speaking a Benkelman Beam rebound value of 1.15 mm was adopted as the design criteria.

## 4.0 LOADING

Access to the James Bay Road has been controlled and every vehicle entering or leaving the project has been counted. The road has been open to the public since 1984. In addition, all the trucks with the exception of southbound logging trucks that have used the road in recent years, were weighed before obtaining access to the territory. As such, the equivalent single axle loads (ESALs) can be calculated more accurately than is normally the case.

Seasonal weight limits were imposed for truck haul on the main access road. Benkelman Beam deflections indicated that the maximum deflections occurred in the August to October period when the subgrade was completely thawed. The limits for this period were the same as the provincial limits - 5,400 kgs (12,000 lbs.) on the steering axle, 10,000 kgs (22,000 lbs.) on a single axle and 17,000 kg (38,000 lbs.) on a tandem axle. The initial fuel haulers were conventional tanker trucks with a steering axle and two tandem axles. A different unit was introduced in 1977 and gradually replaced the initial units. These tankers consisted of a steering axle, a tandem axle and 3 single axles.

These load limits were increased by 10 percent for the May 1 to July 31 hauling period, and by 20 percent for the winter period (November 15 to April 30). The ESAL calculations shown in this report are based on the April to October period as winter traffic was not counted for the ESAL calculations.

In addition, there were a number of very heavy loads such as transformers that weighed approximately 500 tonnes. These were transported using special haul units and are not considered in this study.

## 5.0 DETAILED CRACKING STUDIES

With the extensive use of the 300-400 penetration grade asphalt cement, nine test sections were selected for detailed crack counts in the winter of 1975. These sections were selected on various soil types and in most cases were near maintenance centres for ease of measurement. The sections were each 1.6 kilometres in length. The cracks were counted in October of 1975, March of 1976, June 1978, May 1980 and for those sections still in existence in May 1999. Table 5 contains information from the nine original test sections.

**Table 5 : Number of Transverse Cracks in Test Sections of James Bay Access Roads Paved in 1975**

| SECTION | FROM KM | TO KM | SOIL TYPE | NUMBER OF TRANSVERSE CRACKS | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | 1975 | 1976 | 1978 | 1980 | 1999 |
| 1* | 0 | 1.6 | 3 | 0 | 27 | 33 | 121 | N/A |
| 2 | 12 | 13.6 | 3 | 0 | 12 | 13 | 16 | 69 |
| 3 | 78 | 79.6 | 2 | 0 | 0 | 1 | 8 | 441 |
| 4 | 138 | 139.6 | 2 | 0 | 1 | 7 | 12 | 128 |
| 5 | 170 | 171.6 | 2 | 0 | 3 | 3 | 7 | N/A |
| 6 | 180 | 181.6 | 2 | 0 | 7 | 6 | 5 | N/A |
| 7 | 264 | 265.6 | 3 | 0 | 19 | 22 | 49 | N/A |
| 8** | 582 | 583.6 | 5 | 0 | 27 | 30 | 47 | N/A |
| 9*** | 599.2 | 600.8 | 3 | 0 | 15 | 21 | 78 | N/A |

* This section was paved with 85-100 penetration asphalt cement left in the tanks from a previous job.
** This section experienced intensive off highway hauling during the dam construction starting in 1975. Subgrade was sand and gravel.
***This section experienced intensive off highway hauling during the dam construction. Subgrade was non - granular material.
N/A These sections have been overlaid.

This portion of the study indicated:

1. Test Section 1 paved with 85-100 penetration asphalt remaining in a tank from a previous job cracked more frequently during the first winter than other sections with the same traffic.

2. All sections with the exception of Test Section 3 at km 78 showed some cracking during the winter of 1975/76. The minimum air temperature at Matagami (km 0) was -40.0$^{o}$C according to Environment Canada records. The minimum air temperature at La Grande (km 586) was -41$^{o}$C according to construction records. (The Environment Canada station was not installed until the summer of 1976 at La Grande).

3. Cracks on sections 1 - 7 tended to be large (12.5 mm) and in some cases continued into the base and subbase layers and could be followed into the ditch. (i.e. cracks were not only in the pavement but in the subgrade). Cracks on sections 8 and 9, which were located on massive granular deposits tended to be very fine cracks (hairline). Many of these fine cracks self-healed during the summer months and could not be located in the fall of 1976.

4. There was no apparent reason for the increased cracking on section 7.

Another study conducted independently from the above investigation was reported by Hode Keyser [2] and fortunately contained some of the same sections. This study also reported information on recovered asphalt cement from the test sections, including sections that were paved in 1976. Table 6 contains a summary of Hode Keyser=s work. It also contains results obtained from cores taken in 1999 on two of these same sections to establish the low temperature SHRP performance grade of the original asphalt cement.

**Table 6 : Summary of 1980 Cracking Study** (After Hode Keyser 1980) **With Some 1999 Results**

| | | 1980 Results | | | | 1999 Results | |
|---|---|---|---|---|---|---|---|
| SITE KM | FULL WIDTH TRANSVERSE CRACKS/KM | AGE | ASPHALT CONTENT | RECOVERED PENETRATION | RING & BALL | RECOVERED PENETRATION | SHRP Grading |
| 78 | 0 | 5 | 7.1% | 159 | 43.3 | 83 | -30.8 |
| 93 | 30 | 5 | 5.7% | 112 | 44.4 | 53 | -31.4 |
| 326 | 35 | 4 | 5.7% | 163 | 42.2 | | |
| 332 | 35 | 4 | 5.3% | 143 | 42.2 | | |
| 578 | 10 | 5 | 6.3% | 168 | 42.2 | | |
| 584 NB* | 135 | 5 | 6.6% | 126 | 42.2 | | |
| 584 SB* | 115 | 5 | 6.6% | 126 | 42.2 | | |

* NB = northbound; SB = southbound. This section experienced intensive off-highway vehicle hauling during the dam construction starting in 1975. Subgrade was sand and gravel.

The average penetration value of the original asphalt cement was 317. At kilometre 78, which had no thermal cracks after 5 years of service, the penetration had dropped to 159 after 5 years and to 83 after 24 years. At kilometre 93, which had a moderate amount of transverse cracking after 5 years, the penetration had dropped to 112 after 5 years and to 53 after 23 years.

The low pavement temperature required to prevent cracking (50% reliability) using the SHRP protocols is -30.3 $^{o}$C (Robertson Method) and -31.8 $^{o}$C (LTPP method). The low temperature Performance Grade established from the recovered cores was between -30.8 $^{o}$C and -31.4 $^{o}$C indicating that the asphalt used would have be similar to the material selected using today=s technology. Based on weather records and Robertson=s equation, the low pavement temperature at Matagami in 1975 was -29.9 $^{o}$C.

These two cracking studies both also indicate a correlation between the amount of thermal cracking and the subgrade soil type and/or the amount of traffic loading. Based on studies done in 1978 and 1980 to order crack sealing materials for 100 kilometres of roadway for the northern section of the highway, the amount of transverse cracking was back-calculated and is summarized in Table 7 for sections built on granular deposits and non-granular deposits for both highway traffic and for intensive truck traffic related to the dam construction. For purposes of this table, Agranular soils@ refers to two large deep sand and gravel deltaic deposits between km 480 and km 520 and between km 580 and km 600. ANon- granular@ refers to all other sections which were composed mainly of rock, muskeg and till. AHighway@ traffic refers to truck traffic between Matagami and La Grande, which was in the order of 200 vehicles per day. AIntensive Truck Traffic@ refers to the haul of granular materials from the pits at km 582 to the dam site at km 620. The average daily truck volume was in the order of 2,000 vehicles per day. Vehicles for this haul included off-highway haulers capable of carrying 120 tonnes and tandem gravel trucks that often transported in excess of 60,000 kilograms, more than double the legal limit.

Table 7 indicates that:

1. There was over a 3-fold increase in the number of transverse cracks between 1978 and 1980 in the non-granular sections. There was an increase of between 30% and 80% in the number of transverse cracks in the same two-year period for sections on granular materials.

2. In 1978 there were 2.5 times as many cracks in granular sections than there were in the non-granular sections for both haul conditions. This observation is consistent with that of other investigators such as Deme [5] who indicated that the number of transverse cracks/kilometre was significantly higher in sand subgrade sections on the Ste. Anne Test Road.

3. Transverse cracking was twice as frequent on the intensively loaded sections than on highway traffic sections for both granular and non-granular soils in both 1978 and 1980.

**Table 7: Comparison of The Number of Transverse Cracks for Highway Loadings and Intensive Construction Loadings for Granular and Non-granular Soils on the James Bay Access Road**

| **SURVEY YEAR** | **HIGHWAY LOADINGS** | | **INTENSIVE CONSTRUCTION LOADINGS** | |
|---|---|---|---|---|
| | NON-GRANULAR | GRANULAR | NON-GRANULAR | GRANULAR |
| **AVERAGE NUMBER TRANSVERSE CRACKS / KM** | | | | |
| 1978 | 11.55 | 28.9 | 26.15 | 64.63 |
| 1980 | 44.16 | 38.8 | 84.50 | 104.60 |
| | | | | |
| 1980/1978 RATIO | 3.82 | 1.34 | 3.23 | 1.82 |
| **RATIO OF CRACKS IN GRANULAR / CRACKS IN NON-GRANULAR** | | | | |
| | HIGHWAY LOADING | | INTENSIVE LOADING | |
| 1978 | 2.50 | | 2.47 | |
| 1980 | 0.88 | | 1.24 | |
| **RATIO OF CRACKS INTENSIVE CONSTRUCTION LOADING / CRACKS HIGHWAY LOADING** | | | | |
| | NON GRANULAR | GRANULAR | | |
| 1978 | 2.26 | 2.24 | | |
| 1980 | 1.91 | 2.37 | | |

# 6.0. LONG TERM PERFORMANCE TRENDS

## 6.1 Distress Surveys

Distress surveys have long been used to plan and implement pavement management strategies and a system was developed for the James Bay Access Road in 1975. It was based on the visual inspection of distresses due in part to the James Bay Road's isolated location and the problems of obtaining and operating sophisticated equipment in such a region. Sections were selected based on terrain type and paving contract lengths with the overall consideration that the minimum length of a section was 20 kilometres as it would be impractical to mobilize rehabilitation equipment for anything less. The inspection system measured ride score and 8 distresses. As different distresses occurred in the field they were added to the system and in 1999, 16 different distress types along with ride score were catalogued. The original and additional distresses are indicated in Table 8.

The evaluations consisted of a visual inspection of the roadway surface by a panel of experienced highway engineers. The severity of the distress was rated from 0 (no distress) to 4 (very severe distress) and the extent was rated from 0 (no distress) to 4 (distress throughout). Table 9 is a summary of rating criteria.

**Table 8: Roadway Surface Distress Weighting Values**

**Original Distresses 1975 study** *Additional Distresses added to the study after 1975*

| DISTRESS | WEIGHTING VALUE $w_i$ |
|---|---|
| *Ravelling* | 3.0 |
| **Bleeding** | 0.5 |
| **Patching** | 1.0 |
| **Rutting** | 3.0 |
| **Distortions** | 3.0 |
| *Longitudinal Wheel Track - Single Cracks* | 1.0 |
| **Longitudinal Wheel Track - Alligator Cracks** | 3.0 |
| **Centreline - Single Cracks** | 0.5 |
| *Centreline - Alligator Cracks* | 2.0 |
| *Pavement Edge - Single Cracks* | 0.5 |
| *Pavement Edge - Alligator Cracks* | 1.5 |
| **Transverse - Single Cracks** | 1.0 |
| *Transverse - Alligator Cracks* | 3.0 |
| *Longitudinal Meander and Midlane Cracks* | 1.0 |
| *Block Cracks* | 1.5 |
| **Potholes** | 1.0 |

The ridescore or RCI (Ride Comfort Index) is evaluated on the scale of 0 to 10 by the rating panel. A ride of 8 is typical of a new pavement and few sections have ridescores of less than 3. They are usually reverted to gravel if the ride becomes this severe.

The results shown below are slightly biased. The 1996 and 1999 ratings rated only original pavements. If portions of a section had been patched or replaced, the performance of the patched sections was not included in the ratings. In previous years, the patched sections were included in the ratings resulting in improvements in the ratings for the distresses that had been repaired. Also, by omitting the sections that had been patched in 1996 and 1999, the performance of these sections is somewhat overestimated in that the worst portions of the section have been repaired. Notwithstanding these cautions, the basic trends are still evident.

**Table 9: Summary of Rating Criteria for Roadway Surface Distresses**

**SEVERITY**

| DISTRESS | VERY SLIGHT | SLIGHT | MODERATE | SEVERE | VERY SEVERE |
|---|---|---|---|---|---|
| CRACKING (width) | Hairline < 2 mm | 2 to 10 mm | 10 to 20 mm no spalling | 20 to 25 mm spalling | > 25 mm multiple or start of alligatoring |
| RUTTING (depth) | < 5mm | 5 to 10 mm | 10 to 15 mm | 15 to 25 mm | > 25 mm |

**EXTENT**

| DISTRESS | FEW | INTER-MITTENT | FREQUENT | EXTENSIVE | THROUGH-OUT |
|---|---|---|---|---|---|
| ALL DISTRESSES EXCEPT TRANSVERSE CRACKS * | < 5% | 5-20% | 20-50% | 50-80% | > 80% |
| DISTANCE BETWEEN TRANSVERSE CRACKS | >30 metres | 15 to 30 metres | 10 to 15 metres | 3 to 10 metres | < 3 metres |

### 6.1 Composite Pavement Condition Indices

The following paragraphs describe deterioration in terms of individual distresses. However, pavements are seldom rehabilitated based on the deterioration of individual distresses. Rather a combination of distresses results in the need for rehabilitation either to preserve the asset or to improve the serviceability (ride score). Composite Indices have developed over the years, which combine distress extent and severity factors with ride score. These indices are particularly useful for comparing different sections of highway and for predicting the long term performance of roadway sections.

The original rating system was based on a procedure used in the state of Washington [6] and was modified slightly to reflect Canadian conditions. The original composite index used was suggested by Leclerc [7].

$R_R$= ((10 x RCI) x (100 - sum of distress penalties))$^{0.5}$

where : $R_R$ = Road Rating
RCI = Ridescore

The rating appeared to work very satisfactorily with the first five years of data, but as pavements became more severely distressed, the composite rating went off the end of the scale into negative values. Consequently a system developed by the Ontario Ministry of Transportation [8] was adapted to the distresses found on the James Bay Access Road. The PCI is defined as:

$$PCI = 100(0.1 \times RCI)^{0.5} \frac{(221 - DMI)}{221} \times c + s$$

where: PCI = Pavement Condition Index
RCI = Ride Comfort Index (ridescore)
DMI = Distress Manifestation Index
221 = Maximum value of the DMI
c,s = Calibration constants
c=0.924 and s = 8.856

$$DMI = \sum \text{Distress Factors}$$

Distress Factor = $w_i(s_i+d_i)$
where: $w_i$ = weighting value representing the relative weight of each distress in relation to other distresses (Table 8)
$s_i$ = Severity of distress on a scale of 0 to 4
$d_i$ = Density of distress occurrence on a scale from 0 to 4.

In general terms, a highway with a PCI of 73 or greater is in very good condition, a highway with a PCI of 68 to 73 is in good condition, highways with a PCI of 63 to 68 are in fair condition, a highway with a PCI between 55 and 63 (poor) needs an overlay and highways with PCI's less than 55 are in very poor condition and need extensive repairs before an overlay [9].

### 6.1.1 Pavement Condition Index

Figure 2 indicates the relationship between the PCI and age for the five soil types. The performance of pavements on the very weak soils (Soil 1) was very poor. The performance of pavements constructed on Soils 2 and 3 were similar. The performance of pavements on good soils (Soil 4) and excellent soils (Soil 5) were also similar. Using an average Aoverlay trigger value@ of a PCI of 60, the approximate life of the pavements before an overlay is required is shown in Table 10.

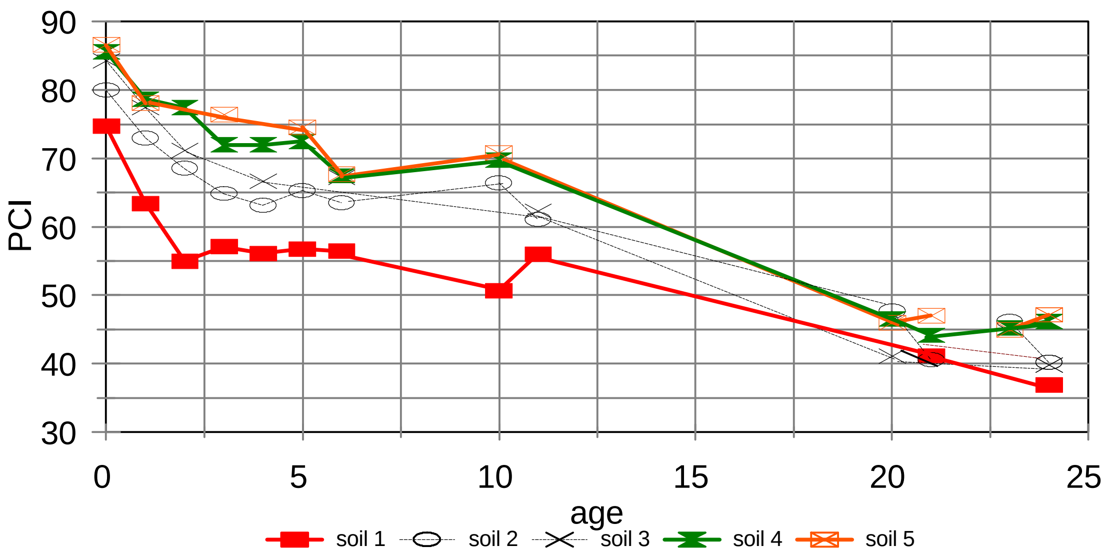


**FIGURE 2 : EFFECT OF SOILTYPE ON PCI (PAVEMENT CONDITION INDEX) OVER TIME**

**Table 10: Service Life of Pavements Before Overlay Required (at PCI = 60)**

| SOIL TYPE | SERVICE LIFE TO OVERLAY, Years |
|---|---|
| 1 | 3 |
| 2 | 12 |
| 3 | 12 |
| 4 | 14 |
| 5 | 14 |

### 6.1.2 Ridescore

The reason for the poor performance of Soil 1 pavements is clearly evident in Figure 3. After only two years of service, the ridescore has deteriorated to 4.0 due to major distortions in muskeg areas and the melting of ice lenses in the permafrost areas. Most Soil 1 pavements were replaced before the evaluation was done in year 20 and only one section has survived to 1999.

Figure 3 also indicates that pavements on Soil 4 and Soil 5 generally had better ridescores over the first 20 years than pavement on Soil 2 and Soil 3 but after 20 years the ridescore on all types was the same (poor).

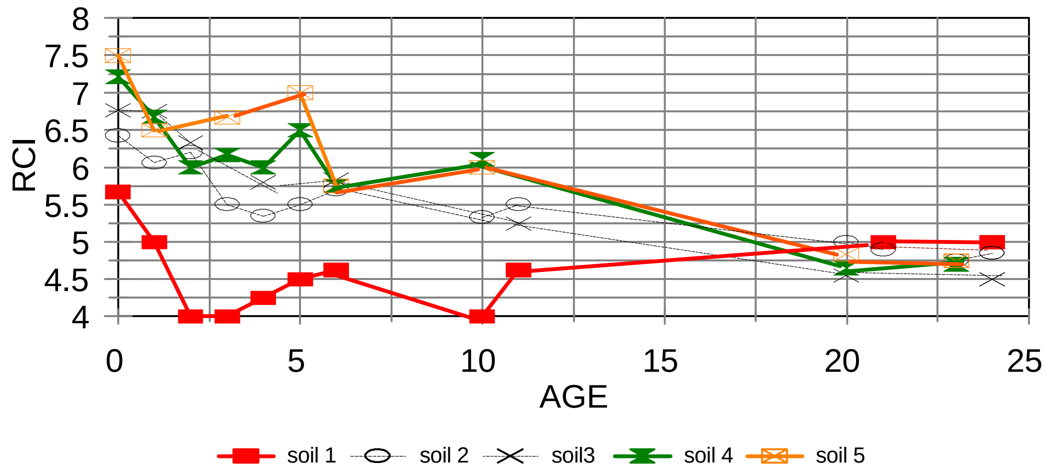


**FIGURE 3: EFFECT OF SOIL TYPE ON RCI (RIDE SCORE) OVER TIME**

### 6.1.3 Rutting

One of the major concerns when the softer grade of asphalt was selected was rutting. Figure 4 shows the relationship between accumulated ESALs and rutting performance. Ruts on pavements on Soil 1,2, and 3 ranged from 10 to 15 mm (moderate rutting) after 200,000 repetitions. Rutting remained at this level until

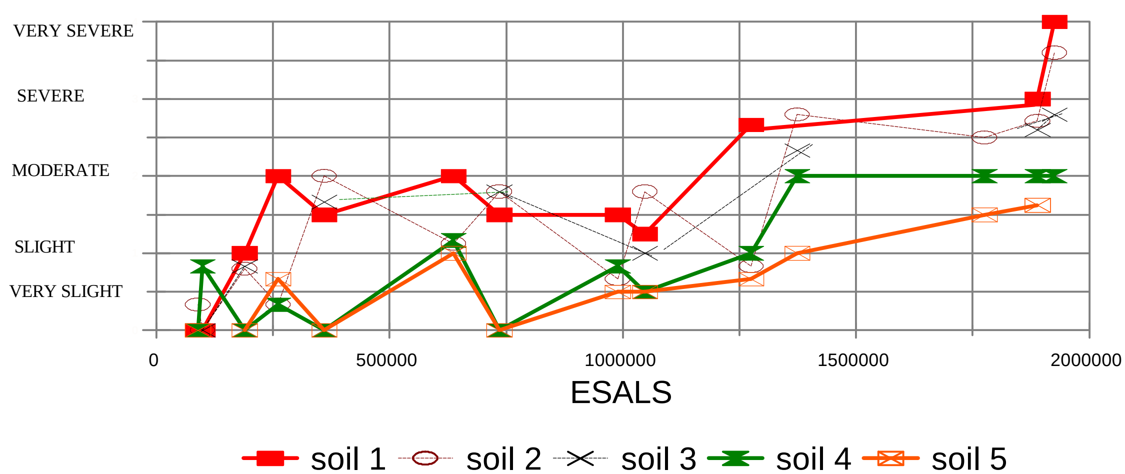


**FIGURE 4: EFFECT OF EQUIVALENT SINGLE AXLE LOADS (ESALs) ON RUTTING SEVERITY**

the ESALs reached 1,250,000 at which point the rutting was rated as severe (15 to 25 mm). For pavements on Soils 4 and 5, rutting was less than 10 mm for 1,250,000 ESAL repetitions. Even after 1,500,000 repetitions, rutting was rated as moderate (10-15 mm) on these soils. For Soil 4 pavements, two showed severe rutting, two showed moderate rutting and two sections showed light rutting after 24 years of service. For Soil 5 pavements, three showed moderate rutting and one showed slight rutting after 23/24 years of service. No attempt has been made to separate rutting in the asphalt layer from rutting in the base, subbase and subgrade layers, but from Figure 4, it is obvious that subgrade type played a significant role in overall rutting performance. Interestingly, the base course and pavement aggregates for most of the Soil 1 pavements were from quarry sources, indicating that the increased angularity from the quarried material did not fully compensate for the weaker subgrade soils.

### 6.1.4 Wheel Track Alligator Cracking

Figure 5 indicates that there was very little alligator cracking in the wheel paths until the pavements experienced 1,250,000 ESAL applications at which time this distress started to appear especially in pavements constructed on weaker soils. Even after 23/24 years only one pavement section on Soil 5 subgrade showed intermittent alligator cracking - all other sections had only a few alligator wheel track cracks. One pavement on a Soil 4 subgrade showed extensive cracking while the other 5 sections had only a few alligator cracks.

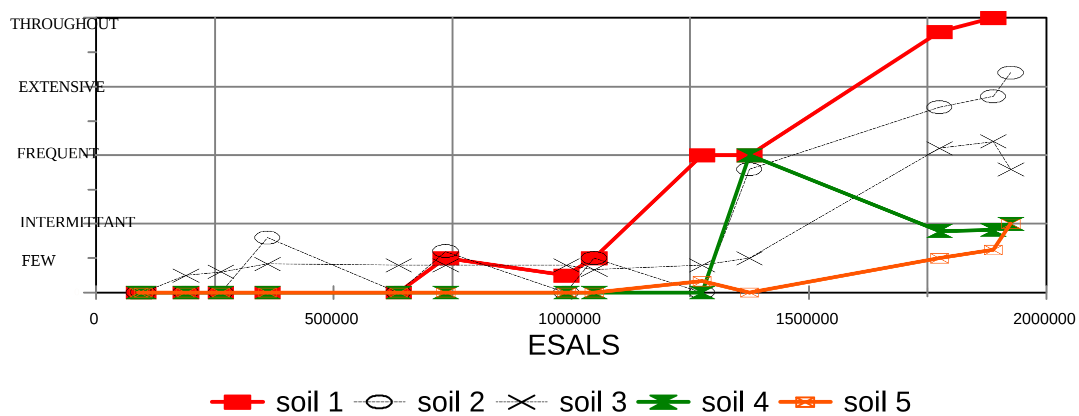


**FIGURE 5: EFFECT OF ESALs ON THE EXTENT OF ALLIGATOR CRACKING**

### 6.1.5 Transverse Cracking

The section on detailed crack surveys indicated cracking patterns at specific locations. Figure 6 indicates transverse cracking on a more generalized 20-kilometre basis. The benefits of using a soft grade asphalt are apparent for the first 10 years of pavement life, particularly as a number of these cracks were located in the subgrade and the pavement would have cracked regardless of the asphalt cement used. Many of the cracks extended into the gravel shoulders leading to the conclusion that the entire subgrade was contracting rather

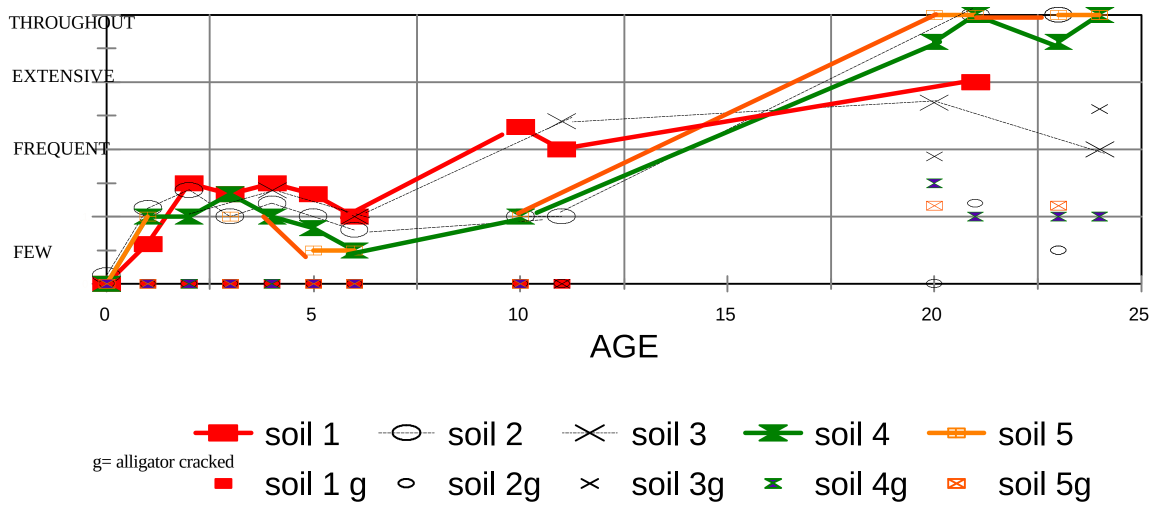


**FIGURE 6 : EFFECT OF SOIL TYPE ON THE EXTENT OF TRANSVERSE CRACKS OVER TIME**

than just the asphalt pavement itself. In February 1976, an inspection of an unpaved section between km 574 and km 581 indicated that there were 10 full width transverse cracks in the base course. After 20 years, all pavements were transverse cracked throughout.

Essentially all sections irrespective of soil type had average transverse crack spacing of greater than 30 metres after one year. Lefebvre [1] reported average first year crack spacing of 130 m for pavements constructed in 1975 and 210 m for pavements constructed in 1976. For soils 2, 4 and 5 this average spacing remained relatively constant for at least the first 10 years of pavement life. Pavements on Soil 1 and 3 had, on average, 10-15 metre transverse crack spacings after 10 years. When the single crack and alligator transverse crack spacings are totaled, after 20 years, virtually all sections had crack spacings of less than 3 metres.

Figure 6 also indicates that the transverse cracks had not alligatored for at least 10 years. Alligatored transverse cracking was more frequent on pavements constructed on Class 3 soils after 20 years.

**6.1.6 Longitudinal Centreline Cracking**

The paving contracts required that Athe contractor shall place the bituminous concrete in a single layer of 275 pounds per square yard using two automatic mechanical pavers operating in tandem (staggered)@ in order to prevent a cold joint on centreline and thereby reduce the potential for centreline cracking.

Figure 7 indicates that there were only a few (less than 5% of the area affected) centreline cracks for most soil types for the first 10 years of pavement life. Soil type 1 appeared to have slightly more centreline cracking in the first 10 years. After 20 years, for pavements on the stronger soils (Soil 4&5) there was more centreline cracking (throughout) than on weaker soils where centreline cracking was rated as between intermittent and frequent.

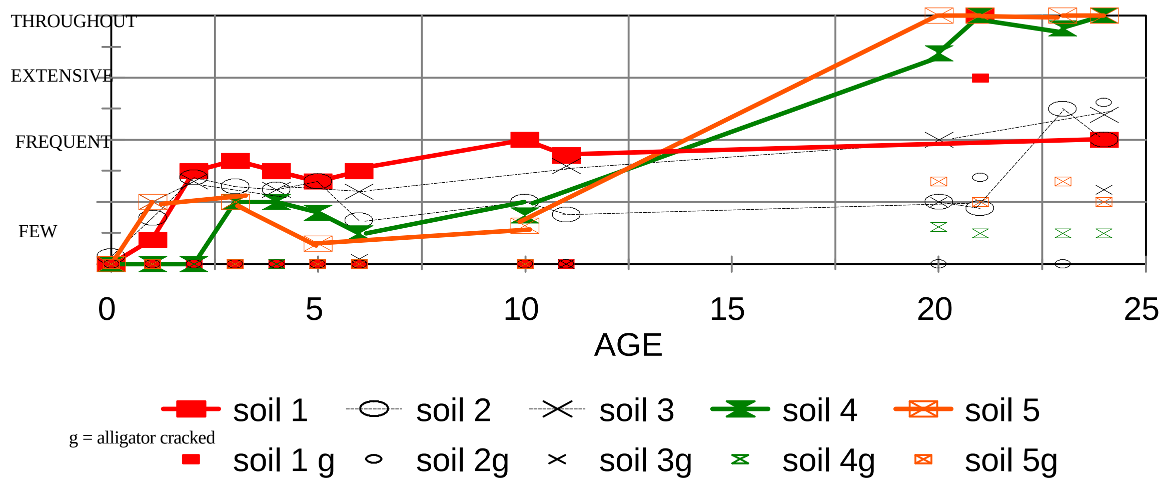


**FIGURE 7 EFFECT OF SOIL TYPE ON THE EXTENT OF CENTRELINE CRACKS OVER TIME**

### 6.1.7 Meander Cracks

Meander cracks are largely associated with distortions, settlements and frost heaves. Figure 8 indicates that there were very few meander cracks in the first 5 years of the pavement life. Meander cracks increased in density as the pavements aged. After 20 years meander cracks were reported throughout.

### 6.1.8 Block Cracks

Block cracks are formed by the intersection of longitudinal and transverse cracks resulting in blocks of approximately 2 to 3 metres square. This type of cracking is normally associated with aged asphalts. Figure 9 indicates that there was no evidence of block cracking in the first 10 years of pavement life. All pavements on granular soils (Soils 4 and 5) were block cracked throughout after 20 years. Pavements on weaker soils (Soil 2 and 3) were slightly less block cracked.

## 7.0 PERFORMANCE OF HEAVILY LOADED SECTIONS

The sections between kilometre 586 and 620 were used to haul 8,000,000 cubic metres of sand and gravel for the construction of the main dam and dykes at the LG-2 power complex. There were no load limits on these sections and the road was used by 40, 70 and 120 tonne off-highway haulers as well as tandem trucks loaded to up to the twice the legal limit. [10]. The sections were not subject to seasonal restrictions.

The typical pavement cross section of this portion of the highway consisted of 100 mm of asphalt concrete, 300 mm of 0-20 mm base course and 300 mm of crushed 0-60 base course. The subbase course thickness

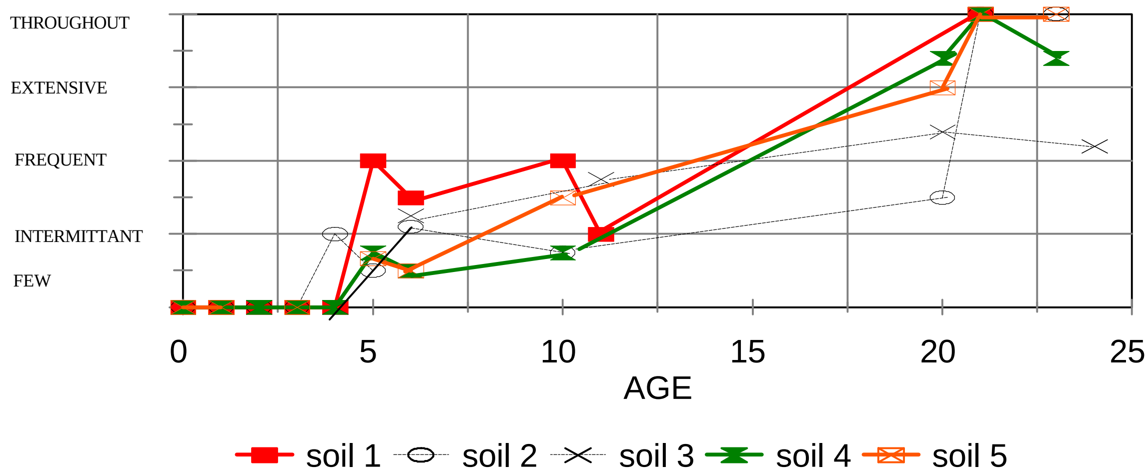


**FIGURE 8 : EFFECT OF SOIL TYPE ON THE EXTENT OF MEANDER CRACKS OVER TIME**

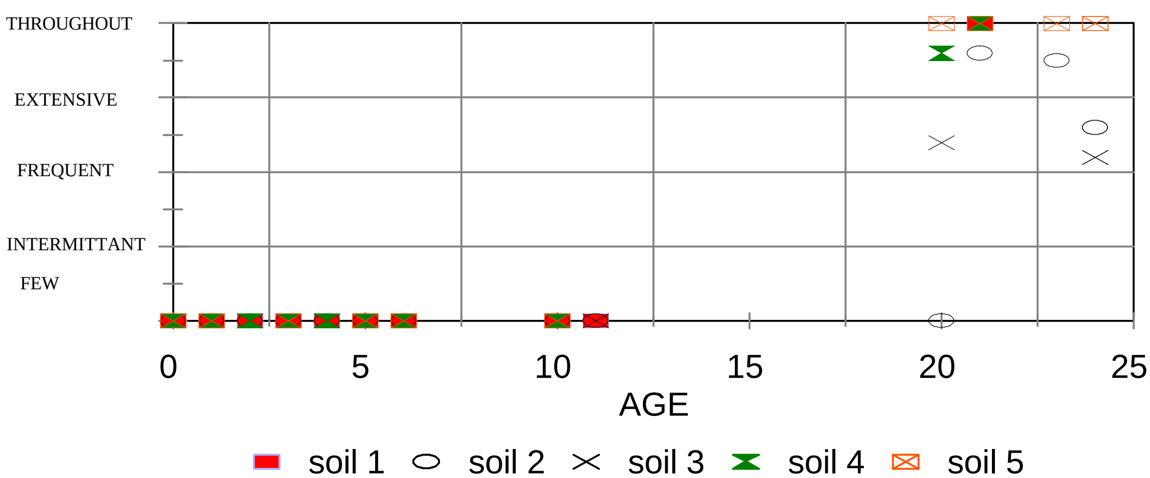


**FIGURE 9 : EFFECT OF SOIL TYPE ON THE EXTENT OF BLOCK CRACKS OVER TIME**

varied depending on soil type similar to the thicknesses used on the main access road. The pavement on this section was 11 metres wide.

Figure 10 compares the data from the LG-2 haul sections and the performance curves developed in this paper for the rest of the highway. It indicates that the pavement on Soil 1 on the intensive haul sections performed even more poorly than soil 1 pavements on the main access road. Pavements on soils 3 to 5 performed similarly to the equivalent pavements on the main access road. There were no soil 2 sections on the intensive haul sections. These sections were all repaved after the completion of the LG-2 dam and dyke construction.

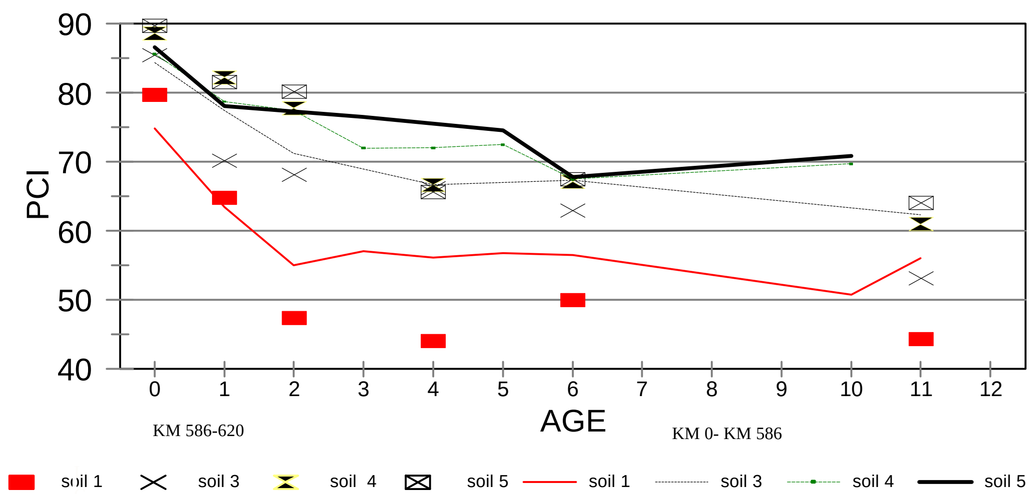


**FIGURE 10 : COMPARISON TO HEAVY HAUL SECTIONS KM 582-KM 620**

## 8.0 OVERLAY PERFORMANCE

A number of pavements have either been overlaid or pulverized and overlaid in the past 7 years. Figure 11 indicates the performance to date of these rehabilitated pavements. In some cases, the pavements were overlaid and in other cases the existing pavement was pulverized before it was overlaid. This accounts for some of the spread in the data, particularly for pavements on Class 3 soils. The overlays used a 200-300 penetration grade asphalt.

## 9.0 SUMMARY AND CONCLUSIONS

The James Bay Access Road was paved in 1975 and 1976 with a 300-400 penetration grade asphalt cement. The asphalt selection was designed to minimize thermal cracking rather than to prevent it completely. The use of even a softer grade of asphalt cement was ruled out because of concerns of rutting, given the truck traffic that was anticipated. The design temperature of the pavement was -40 $^{0}$C.

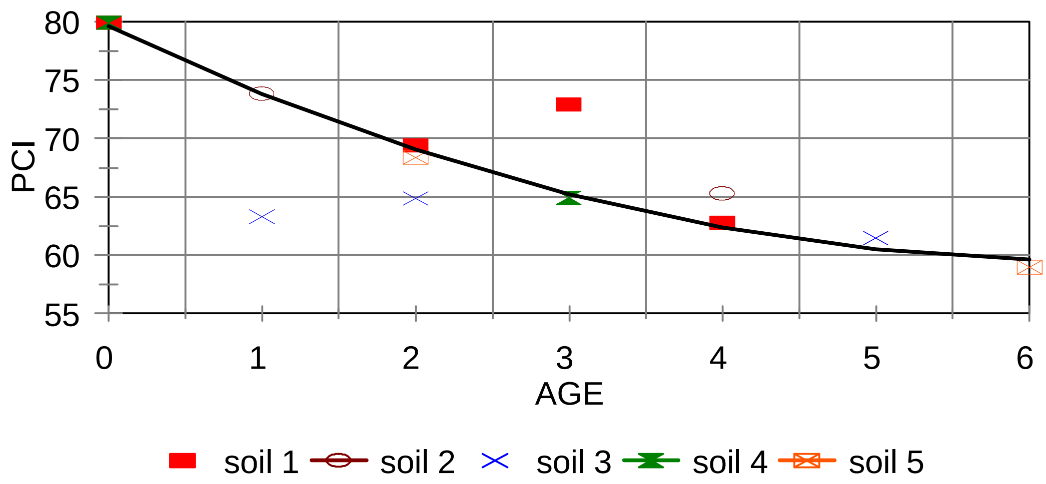


**FIGURE 11 : EFFECT OF SOIL TYPE ON PERFORMANCE OF REHABILITATED SECTIONS**

During the winter of 1975/76, the minimum air temperature was -40°C at Matagami and -41°C at La Grande. Essentially all sections had average transverse crack spacing of greater than 30 metres after the first year. Average first year crack spacings of 130 m were reported for pavements constructed in 1975 and 210 m for pavements constructed in 1976.

For a comparison with current technology the Performance Grade Asphalt Cement required for these for these locations was calculated to be PG 46-34 at 50% reliability and PG 52- 40 at 98% reliability. The pavement was sampled at two locations in 1999 and the Performance Grade low temperature ranged from -30.8°C to -31.4°C

Nine test sections were selected for detailed crack counts in the winter of 1975. This portion of the study indicated that:

1. The section paved with 85-100 penetration grade asphalt cement that remained in a tank from a previous job cracked more frequently during the first winter than other sections with equivalent traffic.

2. All sections with the exception of Test Section 3 at km 78 showed some cracking during the winter of 1975/76. Test Section 3 had no transverse cracking in its 1.6 kilometres length.

3. Cracks on rock and till subgrade sections tended to be large (12.5 mm) and in some cases continued into the base and subbase layers. Cracks located on massive granular deposits were more frequent and tended to be very fine or hairline cracks.

4. After three years, there were 2.5 times as many cracks in granular sections than there were in the non-granular sections for equivalent traffic conditions.

5. Transverse cracking was twice as frequent on sections that were subject to intensive dam construction traffic for both granular and non-granular soils.

An analysis of thirty 20 kilometre long sections selected for pavement management purposes indicated that;

1. Subgrade soil type strongly influenced ride score, wheel track alligator cracking, rutting and the overall pavement condition index.

2. Transverse and longitudinal cracking densities remained relatively constant for the first six years. Cracking densities tended to increase by year 10 to moderate levels and by age 20, the pavements were cracked throughout.

3. Meander crack did not appear for five years and remained relatively minor for pavements between 5 and 11 years old. Most pavements had meander cracks throughout by age 20.

4. There was no block cracking in the first 11 years of pavement life but most pavements on granular subgrades were block cracked throughout by year 20.

***The opinions expressed and conclusions presented in this paper are those of the authors and do not necessarily reflect the official views of their agencies.***

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