Behaviour of road bases stabilized with cement and asphalt in a cold climate

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Presentation prepared for the session:
In- situ and laboratory accelerated pavement tests

of the 2004 Annual Conference of
the Transportation Association of Canada
Québec (Québec), September 19-22, 2004

Summary

A France-Quebec collaborative project between Laboratoire Central des Ponts et Chaussées (LCPC) and the Ministère des Transports du Québec (MTQ) led to the design, construction and rigorous monitoring of the behaviour over time of four experimental road sections set up in Quebec. The project's main objective is the validation and improvement of French and Quebec pavement design methods under severe frost conditions, and more specifically, the calibration of the damage models based on the distress observed on the experimental sections. The design was planned to observe damage over a short period (three years).

This article presents a description of the experimental site and the data collection program put in place in 1998 to monitor the performance of the test sections and the results to date, shedding light on the differences in behaviour, in a harsh climate, between a hot mix asphalt pavement and a pavement with a cement-treated base.

From the first year, the cement-treated test beds showed transverse cracks corresponding to the precracks and wheel path cracks related to fatigue damage, while the hot mix asphalt test beds show no cracks after five years.

In the Quebec climatic context, the use of concrete stabilized with a high percentage of cement offers few advantages compared to a conventional flexible pavement. Stabilization of bases composed of new or recycled materials with a mixed binder composed of an emulsified asphalt and a small quantity of cement offers strong potential for use on high-traffic roadways.

1. Introduction

Under severe frost conditions, a pavement structure should be designed to withstand the stresses transmitted under the effect of traffic and satisfy the criteria of protection against the effects of frost and thaw. Frost can have considerable effects: heat shrinkage cracking of the bound materials, frost heaving and loss of bearing capacity during thaw (1), (2), (3), (4) and (5). Given the evolution of design methods and the absence of extreme winters in France, the Laboratoire Central des Ponts et Chaussées (LCPC) from France and the Ministère des Transports du Québec (MTQ), in 1998, undertook a collaborative project with the objective of optimizing their freeze-thaw pavement design method (6) and (7). Two types of pavements – pavements with cement-treated bases and hot mix asphalt pavements – were selected for construction of an experimental site in Quebec.

The main objectives of the collaborative project are as follows:

- Validate, refine and compare the French and Quebec design methods with regard to the frost resistance of pavements subjected to harsh winters (8) and (9);
- Forecast the propagation of frost and thaw phenomena and the associated swelling based on the thermal characteristics of the pavement materials and the soil, depending on the climate (10);
- Specify the fatigue damage models based on the mechanical properties of the pavement materials and the load bearing soil, depending on the traffic, and then calibrate the models to the appearance of distress on the test beds (11);
- Improve in-situ auscultation and instrumentation techniques and interpretation of results;
- Observe the behaviour of pavements with a precracked cement-treated based under the winter climate and operating conditions peculiar to Quebec.

This paper presents a description of the experimental site and the data collection program put in place in 1998 to monitor the performance of the test sections. The results of the in-situ instrumentation and readings are stated, then analyzed, shedding light on the differences in behaviour, in a harsh climate, between a flexible pavement with hot mix asphalt surfacing commonly used in France and Quebec and a pavement with a cement-treated base commonly used in France and very little used in Quebec.

2. Stabilization of pavements with cement-treated bases (hydraulic binders)

Pavements with bases treated with hydraulic binders (cement-treated bases) thicker than 250 mm and meeting the criteria of the rational dimensioning method, began to be widely used in France starting in 1975 (12). Cement-treated bases are composed of new aggregates less than 20 mm in diameter. They are mixed in a batch mixer and generally contain 3% to 4,5% cement in relation to the mass of dry materials. A setting retardant is also added to the mixing water to increase the workability time and improve implementation. Cement-treated bases are most often covered with a relatively thin layer of hot mix asphalt (60 to 150 mm).

The main distress observed in cement-treated pavement is transverse cracking under the effect of concrete shrinkage and heat shrinkage (12). The French design method accounts for the inevitable presence of these transverse cracks by an increase in stresses at the edge of the cracks. The factors influencing cement-treated base shrinkage cracking are as follows: nature and granulometry of aggregates, nature and bitumen content, implementation period, annual climate, traffic, thickness and nature of surface hot mix asphalt. Uncracked cement-treated bases show spacing between the naturally forming transverse cracks, which ranges between 5 and 10 m. Precracking techniques exist, such as cutting notches in the surface of the course, emulsified asphalt joints and corrugated joints, which allow location and control the opening of the cracks at intervals of less than 5 m. These precracks are sealed once they appear to the surface of the wearing course. Several other techniques can also delay the rise of cracks in the asphalt driving course, very few of which prove to be effective for pavements with cement-treated bases, given the substantial thermal movements in the joints (13).

A cement-treated base is characterized by its modulus of elasticity, its tensile strength and its fatigue behaviour. The high rigidity of the cement-treated layer considerably limits the vertical compression stresses transmitted to the load bearing soil, provided that the opening and severity of the transverse cracks are controlled. However, pavement with a cement-treated base is especially sensitive to repeated passages of vehicles and fatigue damage (12). The horizontal tensile stress in the lower fibre of the rigid course is crucial in the design of such a pavement. This is why the cement-treated base thicknesses now used in France are greater than 250 mm. In general, to withstand one million load cycles, the ratio of load stress (σ_6) to static tensile strength (σ_0) must be less than 0,50. The useful life of cement-treated base is very sensitive to variation in its thickness and its degree of consistency.

In Quebec, bases stabilized with hydraulic binder under flexible surfacing have been very little used. In addition to the Saint-Célestin experimental site, two experiments with resurfacing and in-situ stabilization of the base with Portland cement were conducted in 1996 and 1998 on roads stressed by heavy vehicles (14) and (15). In both cases, the pulverized materials were composed of the former hot mix asphalt and part of the aggregate base in equal proportions. The quantity of cement incorporated in-situ in the form of grout, ranged from 7% to 9% in relation to the mass of dry materials. Four years after commissioning, several longitudinal cracks were counted, associated with a fatigue rupture of the stabilized material or a weakness of the longitudinal joints between passes of the resurfacing machine. Several transverse cracks (spaced between 10 m and 17 m apart) also appeared in the first years, related to concrete shrinkage and heat shrinkage of the cement material and significantly affecting ride quality. Dynamic load readings showed small deformations compared to a standard flexible pavement but highly variable along the sections. This variability was attributed to the heterogeneity of the resurfaced materials and the difficulty of controlling the flow of injected grout precisely. It was found that the loss of bearing capacity due to thaw remained relatively low (15).

On the MTQ road network, the binders most often used during in-situ resurfacing or at the pavement plant are emulsified asphalts with a small quantity of added cement or lime (0,8 to 1,5% in relation to the mass of dry materials) (16). The cement material makes it possible to reduce the emulsion curing period and increase the structural contribution of the treated materials. The modulus of elasticity of mixed binder-treated bases is generally 2,0 to 5,0 times greater for the same unstabilized materials, depending on the cement content and the stabilized thickness. The addition of cement also significantly improves the mechanical properties under saturated conditions. On the basis of the performance monitoring results, a cement content of less than 1,5% in relation to the mass of dry materials does not lead to concrete shrinkage and heat shrinkage cracking (premature formation of transverse cracks).

3. Description of the experimental site

3.1. Local characteristics

The experimental site was constructed on National Highway 155 in Saint-Célestin, Quebec, Canada. This is a highway with two adjacent lanes. The site is located in the St. Lawrence River plain about 100 kilometres northeast of Montreal (figure 1). Highway 155 was chosen because of the homogeneity of the site, the presence of frost-sensitive soils (silty sand resting on clay) and signs of fatigue damage on the old pavement. It is subject to a harsh climate (average frost index of 1150 °C-day) and bears substantial heavy traffic (750 heavy vehicles/day in each direction). Draft design studies made it possible to characterize the bearing capacity of the subgrade soil, its frost susceptibility level, the drainage conditions, the climate conditions, the density of the forecasted traffic and the properties and composition of the hot mix asphalt and the cement-treated base to be produced (6).

3.2. Test sections

The test beds were constructed by half-roadway, maintaining one lane for traffic. The old pavement was excavated to the desired depth to install the layers constituting the test beds (figure 2). The wearing course for all test beds is hot mix asphalt (HMA) with granulometry of 0/10 mm (EB10S). Test Bed 1 has a base course of HMA with granulometry of 0/20 mm (EB 20). Constructed accordingly to Quebec design method, this test bed serves as a control section. Test Bed 2 has a structure similar to the control section, with thermal insulation added.

The other two test beds, constructed using the French design method, have a cement-treated base (CTB) course. One is thermally insulated (Test Bed 3) and the other is not (Test Bed 4). The thermal insulation was incorporated to separate the effects of traffic from those related to the freeze-thaw cycle of the subgrade soil. The cement-treated base course is made in a batch mixer by mixing its components (crushed aggregate 0-5 mm and 5-20 mm, 3,5 % type 10 Portland cement by weight, 1,9 % hydrated cement stabilization agent by weight and 1,5% added water by weight). The cement-treated base was precracked transversely at intervals of 2,5 m over 50% to 60% of its thickness to control heat shrinkage cracking. The precracks were produced with a saw cut after 10 vibrating roller passes (two third of the compaction power) and the slot was filled with a cationic emulsified asphalt to prevent setting. The precracks that rose to the surface were sealed without filling with a modified asphalt-based product installed hot as soon as they appeared (9).

The thermal insulation used is composed of two layers of extruded polystyrene in plates 25 mm thick. The base course of test beds is composed of granular materials 0/20 mm (MG 20). The bottom course is composed of subgrade sand left in place resting on a supporting mass of natural clay soil.

The constituents of the pavement materials, their manufacturing and their implementation were controlled very rigorously (10) and (11). The controls in relation pavement thermal behaviour focused on:

- the mineralogical nature of the aggregates and the measurement of their maximum density,
- the composition hot mix asphalts EB 10S and EB 20 and particularly their bitumen content,
- the composition of the cement-treated bases and particularly their water, cement and setting retardant,
- in-situ measurement of the density and water content of each pavement course (EB 10S, EB 20, Cement-Treated Base, MG 20, subgrade sand) and the bottom course,
- measurement of the thickness of the pavement courses and the underlying courses.

Initial assessment of the pavement condition was produced one month after commissioning (11). It involved taking core samples of the bound pavement layers. The measurements taken on the core samples validated the thickness and composition of the layers. Destructive coring was also done to validate the thickness of the unbound layers and the position of the extruded polystyrene layers.

4. Data gathering

During construction, each test bed was equipped with a set of sensors for periodic monitoring of the behaviour of the pavement and its different layers (17) and (18). The test beds are equipped with frost depth gauges (progression of the freeze-thaw front), piezometers (position of the water table), thermistors (temperature measurement), Time-Domain Reflectometry (TDR) sensors (volume weight of water content) and frost heaving benchmarks. On the uninsulated hot mix asphalt and cement-treated base test beds, multi-level deflectometers are added to measure the deflection of each layer (19). The uninsulated cement-treated base test bed was also equipped with a longitudinal and transverse deformation gauge at the base of the bound layer (20). Sensors linked to a continuous data acquisition system were also added under Test Bed 1 to measure the water content in the different layers (Theta-Probe) and the temperatures (thermistors) to a great depth (10,5 m). A weather station and a weigh-in-motion station complete the site's instrumentation.

A detailed annual program of readings and observations of test beds behaviour has been in progress since commissioning. It includes pavement structure deformability measurements in the summer, fall, frost and recovery periods, performed with several devices: Falling Weight Deflectometer (FWD), inclinometer, ovalization, multi-level deflectometers and deformation gauges. Detailed mapping of cracking and measurements of rut depth, International Roughness Index (IRI), frost depth and frost heaving are also performed periodically.

5. Presentation of experimental results

5.1. Heat shrinkage cracking

Transverse cracks appeared only on the cement-treated base test beds (figure 3). These cracks corresponds to the heat shrinkage of the cement-treated base because they appear in winter. They are located to the right of the precracks (figure 4). After five years of monitoring, 75% of the precracks have risen to the surface, in whole or in part, on Test Bed 3 and 95% on Test Bed 4. The total length of the cracks risen to the surface represents about 60% of the 600 m of precracks in the cement-treated base (Test Beds 3 and 4). They appear on part of the pavement and then propagated over the entire width.

Precrack opening is generally 1 to 2 mm. This allows entanglement of the aggregate and assures an adequate load transfer between the precrack edges. The load transfer was estimated with the FWD (21). The results are presented in figure 5. It is noted that the load transfer in most cases is greater than 70% after three years. A decrease in load transfer of about 10% per year is also noted.

5.2. Fatigue cracking

Fatigue cracks appeared only on the cement-treated base test beds (figure 6). The cracks developed previously on the uninsulated Test Bed 4 and more rapidly in Direction 2, which corresponds to the busiest lane. The fatigue cracks represent a total of 254 m on Test Bed 4, compared to only 48 m on the insulated Test Bed 3. The cracks first appeared at the locations with the greatest deflection (sectors B and C) during commissioning of the test beds (figure 7).

The MTQ's design for Test Bed 1 corresponds to the design of an urban national highway. The design accounted for traffic of 7,436 million of Equivalent Single Axle Load (ESAL) and a variation of the Present Serviceability Index (PSI) of a value of 2,0 for a fifteen-year period (8). The LCPC designed the uninsulated test beds so that their damage is visible after three years of traffic (8). A 25% calculation risk was adopted, corresponding to the order of magnitude observed when the first surface distress appeared, during the experiments on the LCPC fatigue test track (installation for real-size accelerated road tests).

The preliminary study performed by the LCPC, based on a high load-bearing loss of the frost susceptible load bearing soil due to thaw (modulus divided by 10) forecast that damage after three years of traffic would reach 1,7 on Test Bed 4 and 0,66 on Test Bed 1. The damage value of 1, corresponding to the appearance of the first distress, thus had to be reached after a year and a half to two years of traffic on the Test Bed 4 (CTB) and after five years on Test Bed 1 (HMA). For the insulated test beds, the damage corresponding to the appearance of the first distress should be reached after four years on the cement-treated base test bed and after twenty-seven years on the hot mix asphalt test bed.

Observation of distress on the test beds shows that the first fatigue distress appeared on Test Bed 4 after one year of traffic and affected the entire length of the test bed after three and a half years. On Test Bed 3, the first distress occurred after one year in Direction 1 and after three years in Direction 2. However, this distress only affects 25% of the length of the wheel path length after five years. Finally, no fatigue distress appeared on Test Beds 1 and 2 after five years.

5.3. Summer ride quality

The longitudinal profile of the pavement, represented by the IRI index, makes it possible to assess the degree of ride comfort and characterize the surface roughness (figure 8). No notable increase in the summer IRI is noted for Test Beds 1 and 2 since commissioning, with an average value of 1,3 m/km. Behaviour is similar in both directions. The initial IRI is greater on the cement-treated base sections than on the hot mix asphalt sections. Moreover, on the cement-treated base test beds, the IRI has increased significantly since commissioning.

5.4. Ruts

Rut measurement is an indicator of pavement structural behaviour. On hot mix asphalt Test Beds 1 and 2, rut depth reaches an average of 7 mm on Test Bed 1 and 6 mm on Test Bed 2 (figure 9). This corresponds to an annual increase of around 1,3 mm. Cement-treated base Test Beds 3 and 4 show the smallest increases with an average rut value of 4 mm after five years, which represents an annual increase of 0,8 mm.

5.5. Resilient modulus of subgrade soil

The readings taken with the FWD were used to analyze the seasonal variation of the resilient modulus of the subgrade soil subject to freeze-thaw cycles. This modulus was back-calculated with the revised CHEVLAY2 linear layered elastic systems program based on the Burminster model, by matching the measured deflection basin to the calculated basin (22). Figure 10 shows the seasonal variation of the resilient modulus of the subgrade soil for Test Beds 1 and 4, for the first year of monitoring (1998-1999). It shows Direction 2, where a silty sand layer is found, resting on clay for all sections. Compared to the value measured in October 1998, the results show a strong variation of the resilient modulus. The resilient moduli of the soil are 5 to 12 times higher in winter than in fall. In the spring, the resilient modulus of the thawed soil diminishes. In the case of the clay, it corresponds to 63% of the fall value (Test Bed 1) while for silty sand, it is at 56 % (Test Bed 1) and 39 % (Test Bed 4) of the reference value outside the thaw period.

5.6. Frost depth, frost heaving and ΔIRI

Pavement frost behaviour can be characterized by the frost depth, frost heaving and the change of surface roughness between summer and winter. Table 1 presents, for the five winters of monitoring, the maximum frost depth measured on the four sections with frost depth gauges, the maximum frost heaving measured at the surface with benchmarks, the difference of IRI between winter and summer (ΔIRI) measured with an inertial profilometer (GMR) corresponding of the average of the two directions. The annual frost index is also included. Because of the very mild winters during the first four years of monitoring, the subgrade soil was very few affected by frost during these periods. However, the winter of 2002-2003 was colder and the frost therefore penetrated more deeply into the subgrade soil considered to be frost susceptible. For the uninsulated Test Beds 1 and 4, the subgrade soil was frozen over a thickness of 55 to 75 cm. On Test Beds 2 and 3, which contain thermal insulation, the frost front passed through 9 to 17 cm of the thermal insulation while remaining in the sand subbase (non frost susceptible granular). The frost depth under cement-treated base Test Bed 4 is slightly less, 10 cm on the average, than on hot mix asphalt Test Bed 1, for the five years of monitoring. Frost heaving remained low during the five years of monitoring. The frost heaving obtained in the uninsulated pavements ranged between 4 and 33 mm, while the insulated pavements were subjected to frost heaving ranging between 0 and 5 mm. The cement-treated base Test Bed 4 shows frost heave similar to Test Bed

1 for the milder winters. However, for the harshest winters (2000-2001 and 2002-2003), frost heaving is greater on Test Bed 1. The winter measurements are taken in March. For the four test beds, ΔIRI has remained below 0,50. In general, the ΔIRI is lower for the insulated test beds.

Table 1 – Measurement of maximum frost depth , maximum frost heave and the ΔIRI

Year	Frost index (°C-day)[(1)]	Frost depth (m)				Maximum frost heaving (mm)				ΔIRI			
		Test Bed 1 (uninsulated HMA)	Test Bed 2 (Insulated HMA)	Test Bed 3 (Insulated CTB)	Test Bed 4 (uninsulated CTB)	Test Bed 1 (uninsulated HMA)	Test Bed 2 (Insulated HMA)	Test Bed 3 (Insulated CTB)	Test Bed 4 (uninsulated CTB)	Test Bed 1 (uninsulated HMA)	Test Bed 2 (Insulated HMA)	Test Bed 3 (Insulated CTB)	Test Bed 4 (uninsulated CTB)
1998-1999	666	1,30	0,79	0,83	1,17	20	0	5	20	0,45	0,13	0,27	0,27
1999-2000	796	1,45	0,83	0,88	1,35	20	4	3	17	0,52	0,21	0,23	0,18
2000-2001	1114	1,54	0,84	0,90	1,42	33	3	1	24	0,37	0,16	0,02	0,14
2001-2002	659	1,12	0,80	0,86	1,05	4	1	3	7	0,19	0,09	0,01	0,16
2002-2003	1286	1,70	0,95	1,03	1,62	23	0	1	8	0,16	0,02	0,05	0,17

(1) The thirty-year average of the frost index (Nicolet weather station) is 1150 °C-day

6. Discussion

6.1. Effectiveness of the precracking technique

The load transfers to the precracks were measured over a three-year period, at different periods of the year and at different air temperatures (figure 5). The season and the time of day when the FWD test is performed significantly influence the measured load transfer. Temperature changes result in horizontal movements in the joints and the blowup of the slabs. It is recommended to perform the tests early in the morning in cool weather when the load transfer is at its lowest and the damage at its greatest (23). The annual load transfer decrease, averaging 10%, can be explained by the season and the temperature at which the FWD test was performed. The precracks were sealed as soon as they appeared on the surface and no visible distress of the edges is noted. Precracking every 2,5 m with emulsified asphalt injection therefore worked correctly because the precrack openings remain smaller than 2 mm and the load transfer is sufficient after three years in most cases. However, three cracks in Test Bed 3 show an insufficient load after three years.

6.2. Ride quality upon commissioning

The initial IRI upon commissioning is significantly greater on the cement-treated base sections (2,1 m/km) than on the hot mix asphalt sections (1,3 m/km). This gap is explained by the difference in the techniques of installing the two types of base courses: use of a grader for cement-treated base and a finishing machine for hot mix asphalt.

Moreover, on the cement-treated base test beds, the summer IRI has increased by an average of 0,22 m/km from the initial assessment on Test Bed 3 and 2,5 m/km on Test Bed 4. This is directly related to the presence of both heat cracking and fatigue cracking on these test beds. The cement-treated base test beds were designed so that fatigue cracking is accelerated. According to the deficiency thresholds set by the MTQ for a national highway, this should be the object of a light intervention (mechanized patching, resurfacing course) when the IRI exceeds 2,5 m/km. The summer IRI stayed at its initial value on the cement-treated base test beds, which show no cracking after five years.

6.3. Winter behaviour

Significant differences in winter behaviour are noted between the cement-treated base test beds used for experimental purposes and the hot asphalt mix test beds commonly used in Quebec. The cement-treated base is affected by concrete shrinkage cracking and thermal cracking, which requires precracking every 2,5 m. These precracks, once they appear to the surface, must be maintained periodically by sealing to avoid their distress. The hot mix asphalt pavement structure shows no heat shrinkage cracks after five years, so maintenance by crack sealing is therefore unnecessary.

The frost depth on the cement-treated base Test Bed 4 is slightly less (about 10 cm) than on the hot mix asphalt Test Bed 1. The thermal conductivity of the cement-treated base is slightly less than the thermal conductivity of the hot mix asphalt (0,92 compared to 1,45 W/mK). The thickness of the cement-treated base is also greater, which gives it a slightly higher insulating power. Frost heaving is generally observed to be a little higher on Test Bed 1 than on Test Bed 4.

Seasonal IRI variations have remained low since the beginning of monitoring for the hot mix asphalt and cement-treated base test beds. The mild winters and the slight depth of frost susceptible subgrade soil can explain the low values and the variability of the ΔIRI from year to year. The ΔIRI values are lower on Test Bed 4 (0,14 to 0,27) than on Test Bed 1 (0,37 to 0,52) for the first three years. However, they are similar for the last two years (0,16 to 0,19). In general, ΔIRI measurements below 1.0 m/km denote a road with little frost susceptibility.

6.4. Thaw behaviour

The seasonal variation study of the modulus of the subgrade soil subjected to frost for the first year of monitoring was affected by a relatively mild winter with a frost index of 666 °C-day compared to the thirty-year average frost index (1150 °C-day). On Test Bed 1, the frost affected the silty sand for its entire thickness (300 mm) and the underlying clay for a short thickness of 70 mm. On Test Bed 4, the frost affected 140 mm of silty sand without reaching the clay. It should be noted that frost heaving was of the same order of magnitude on both test beds. The frost weakening on the resilient modulus of the soil can be compared only for the silty sand. The resilient modulus of the silty sand corresponds to 56% (Test Bed 1) and 39% (Test Bed 4). The greater rigidity of the cement-treated base may explain this 17% difference in weakening of the resilient modulus because the stresses transmitted to the soil are weaker.

7. Conclusion

In 1998, the MTQ and the LCPC undertook a cooperation project with the primary objective of optimizing their respective freeze-thaw pavement design methods. This project led to the construction of four test beds in Quebec. These test beds included experimentation with a pavement structure commonly used in France, pavement with a cement-treated base, in the much harsher context of Quebec's climate.

The mild winters of the last few years have not allowed significant differences in freeze-thaw behaviour to be noted between pavement with a cement-treated base and hot mix asphalt pavement. However, the analysis of the data for the harsher winter of 2002-2003 may show accentuated differences.

From the first year, the cement-treated base test beds showed fatigue damage characterized by wheel path cracks. This had initially been forecasted during design to observe damage within a short time. The hot mix asphalt test bed does not show fatigue cracks after five years. However, the cement-treated base structure is thicker than the conventional hot mix asphalt structure (1060 mm versus 930 mm). The presence of a stabilized base course with a very rigid surface (modulus of elasticity 27000 MPa) does not allow the thickness of the pavement structure to be reduced. On the contrary, this thickness must be increased to limit the tensile stress at the base of the rigid course, which is more sensitive to fatigue damage. Under the conditions of the test site for a fifteen-year design (25% risk), the French design method forecasts that the cement-treated base should be 275 mm thick.

The technique of precracking the cement-treated base with emulsified asphalt injection at intervals of 2,5 m, which was used to locate and control crack opening, proved to be effective, as shown by the measurements of load transfer to the precracks and the crack openings of less than 2 mm. However, the faster appearance of these precracks to the pavement surface necessitates regular maintenance in the form of sealing with an hot bituminous product to limit spalling of the edges and the infiltration of water and brine into the pavement structure. In comparison, the hot mix asphalt Test Bed 1 constructed accordingly to Quebec design method does not show any transverse cracks after five years. This performance eliminates the need of maintenance operations and the reduction of ride quality that accompanies the increase in the severity level of cracks under the effect of traffic. The introduction of PG classes for asphalts adapted to the weather conditions of the different regions of Quebec has allowed a considerable reduction in transverse cracking on hot mix asphalt surfaces in Quebec.

In the light of these observations and in the Quebec climatic context, the use of a concrete base stabilized with a high percentage of cement offers few advantages compared to a conventional flexible pavement. A flexible pavement that conforms to the MTQ's structure design criteria, with a sufficient thickness of non-frost susceptible materials to procure partial protection against frost, makes it possible to limit the effects of frost on surface roughness (outside major differential frost heaving zones) and reduce the losses of bearing capacity of subgrade soil due to thaw. Moreover, stabilization of bases composed of new or recycled materials with a mixed binder composed of an emulsified asphalt and a small quantity of cement offers strong potential for use on high-traffic roadways. The addition of cement significantly improves the mechanical properties, primarily in the short term (reduction of emulsion curing time) under saturated conditions. A cement content of less than 1,5% does not result in transverse shrinkage cracking.

8. Acknowledgments

The authors wish to thank La Commission Permanente France–Québec for its invaluable financial support to the project.

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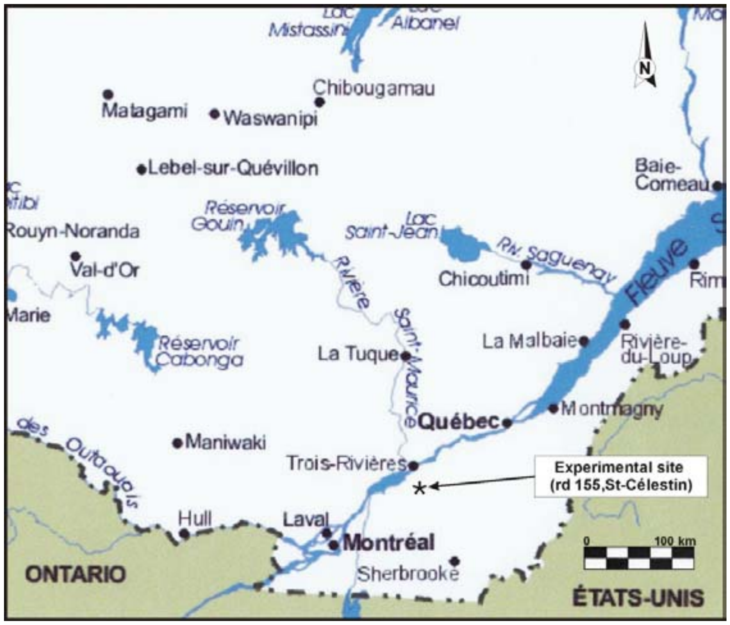


Figure 1 – Site location

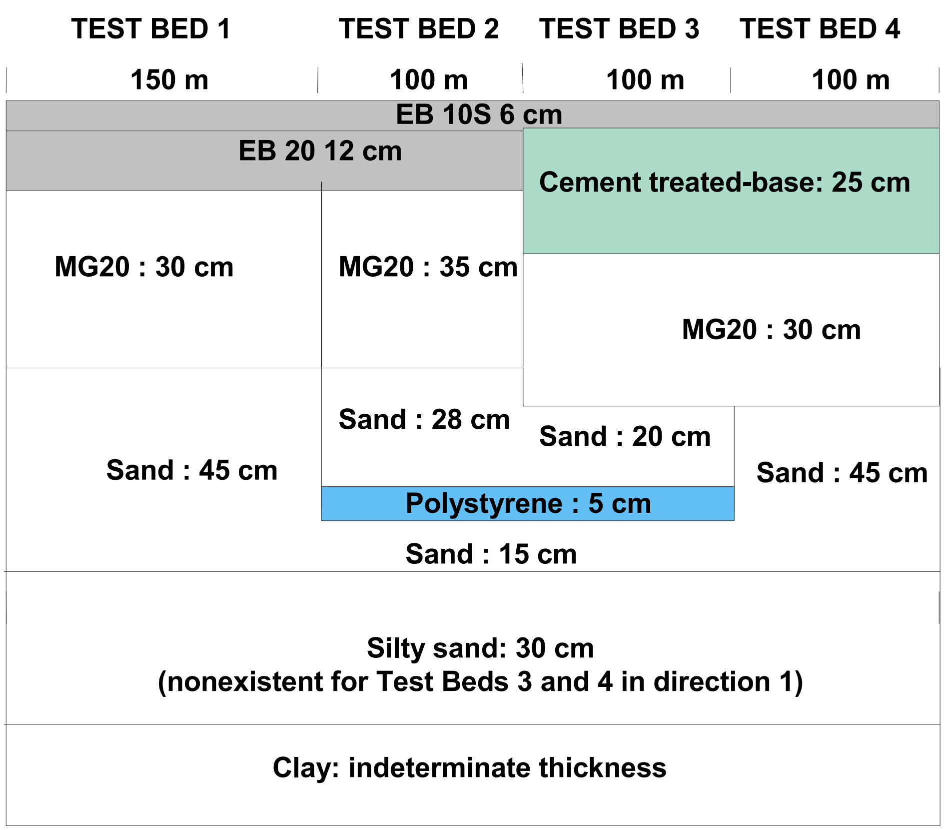
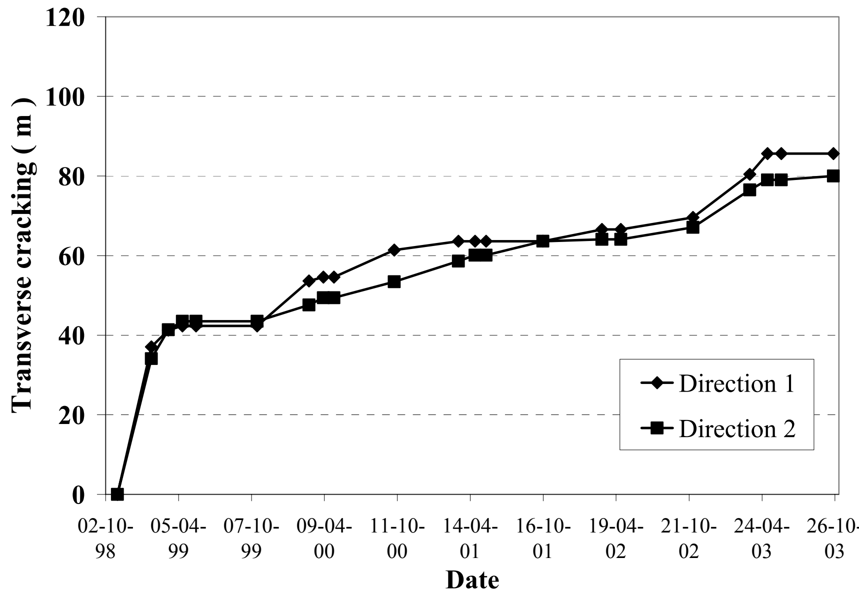
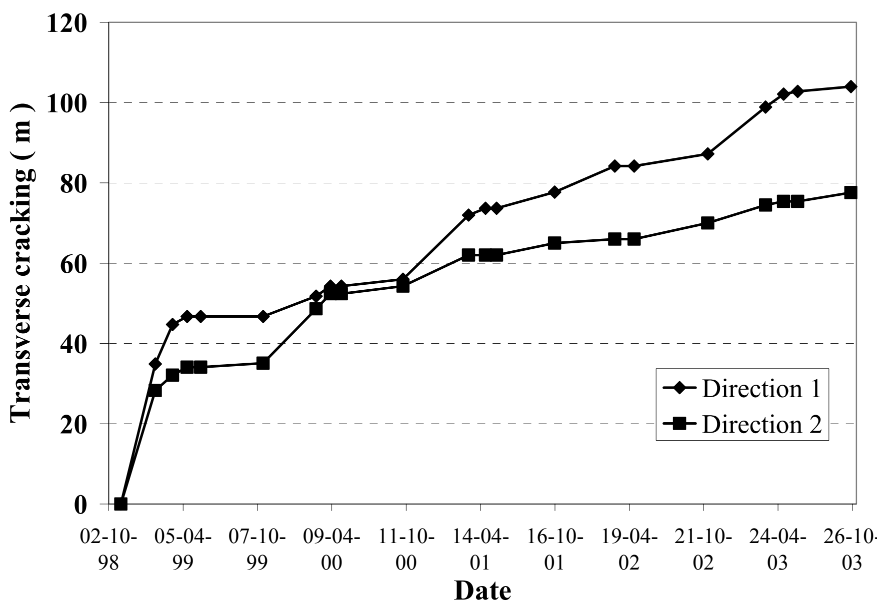


Figure 2 - Composition of the test beds

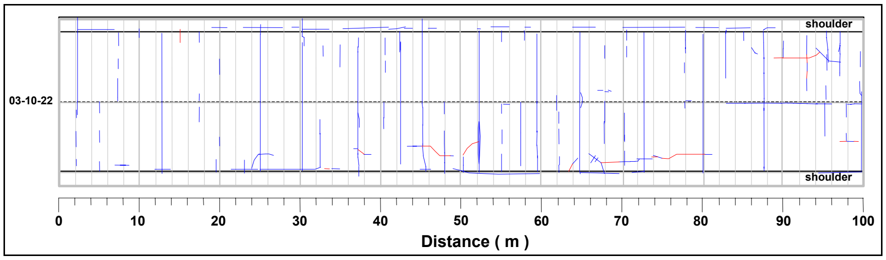


a) Test Bed 3 (insulated CTB)

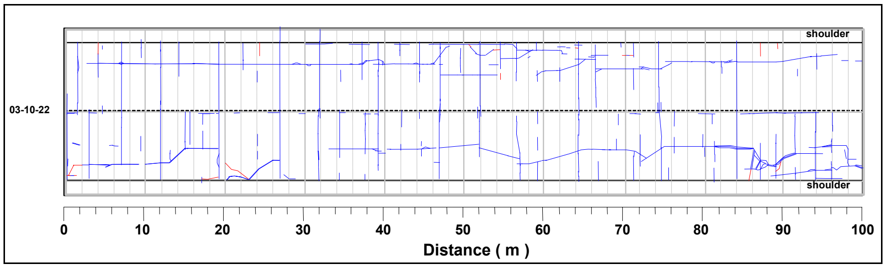


b) Test Bed 4 (uninsulated CTB)

Figure 3 – Heat shrinkage cracking trend



a) Test Bed 3 (insulated CTB)



b) Test Bed 4 (uninsulated CTB)

Figure 4 – Crack mapping (10.31.2002)

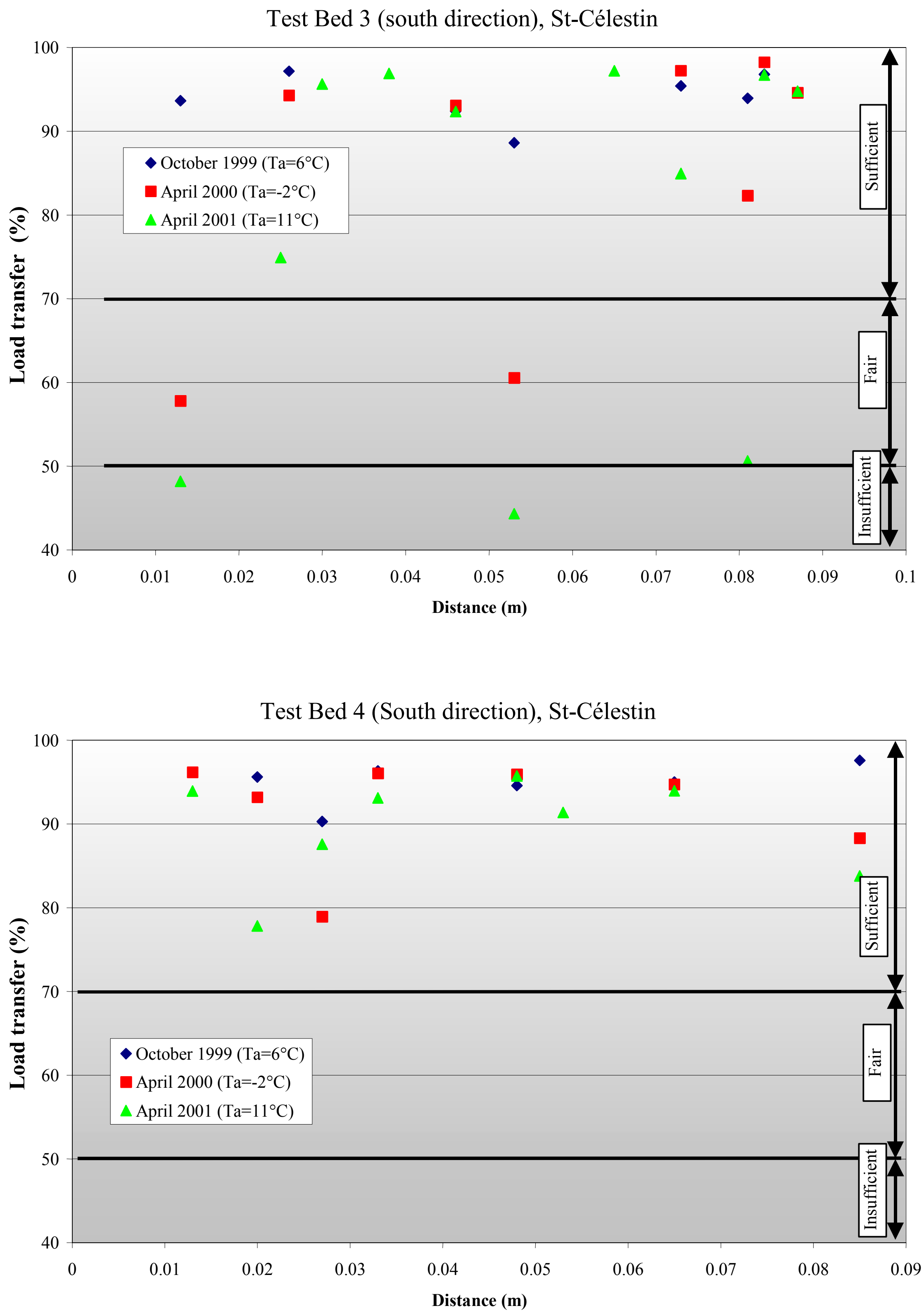
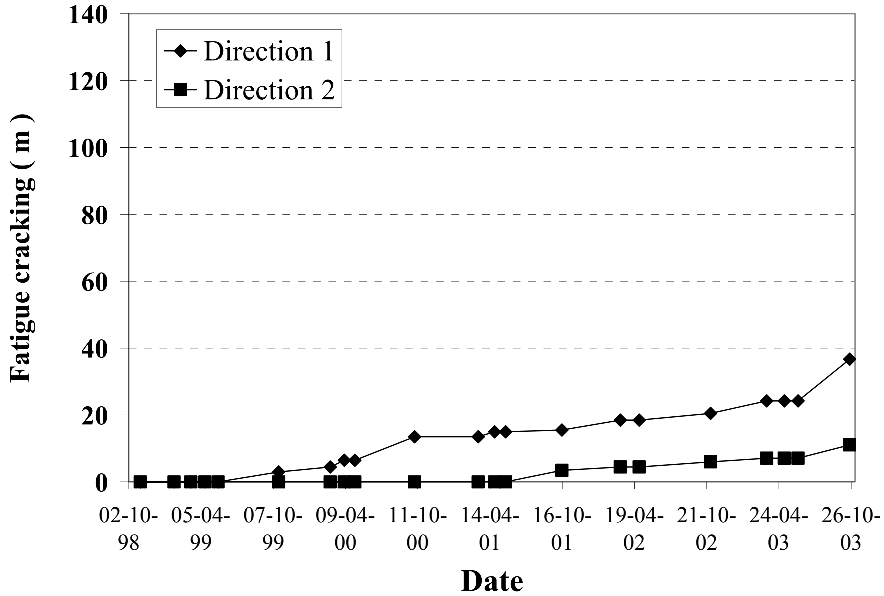
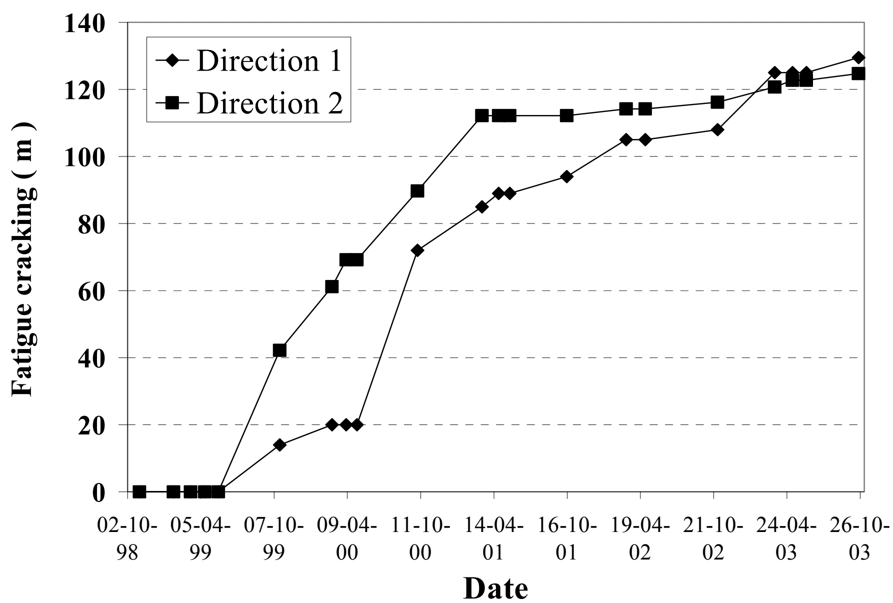


Figure 5 – Percentage load transfer to cement-treated base precracks



a) Test Bed 3 (insulated CTB)



b) Test Bed 4 (uninsulated CTB)

Figure 6 – Fatigue cracking trend

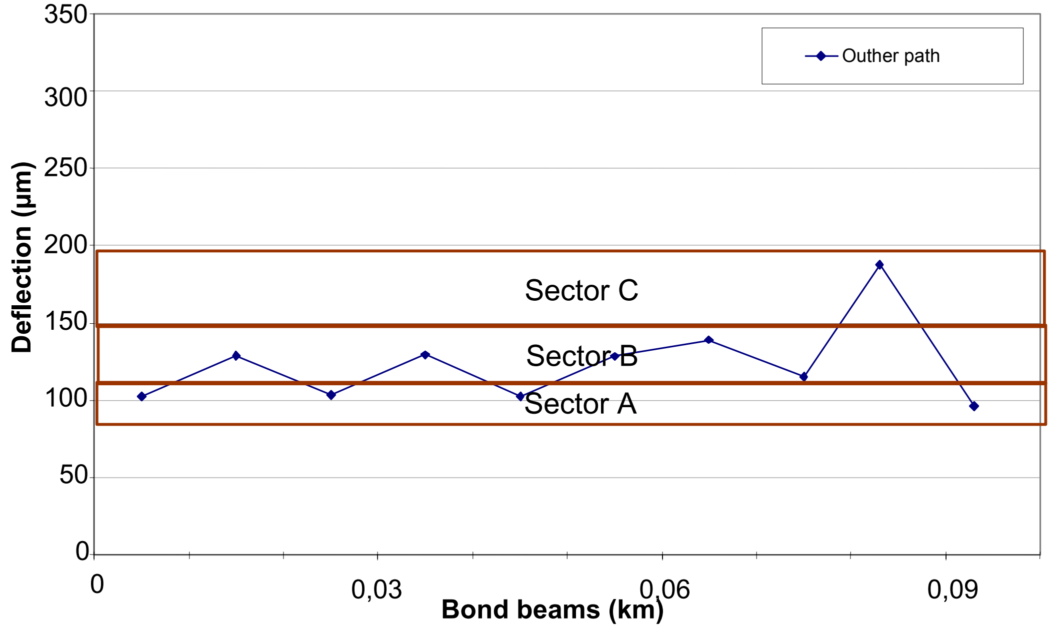


Figure 7 - Deflections (D_0) measured on Test Bed 4 (Direction 2) during commissioning

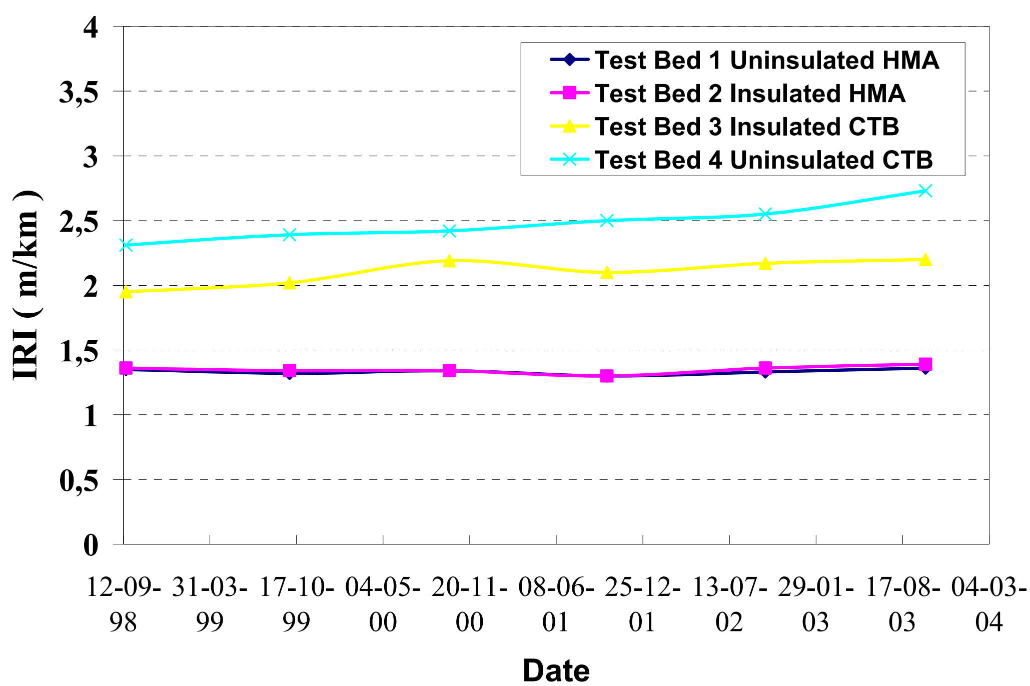


Figure 8 – Ride quality trend in summer period (Direction 1)

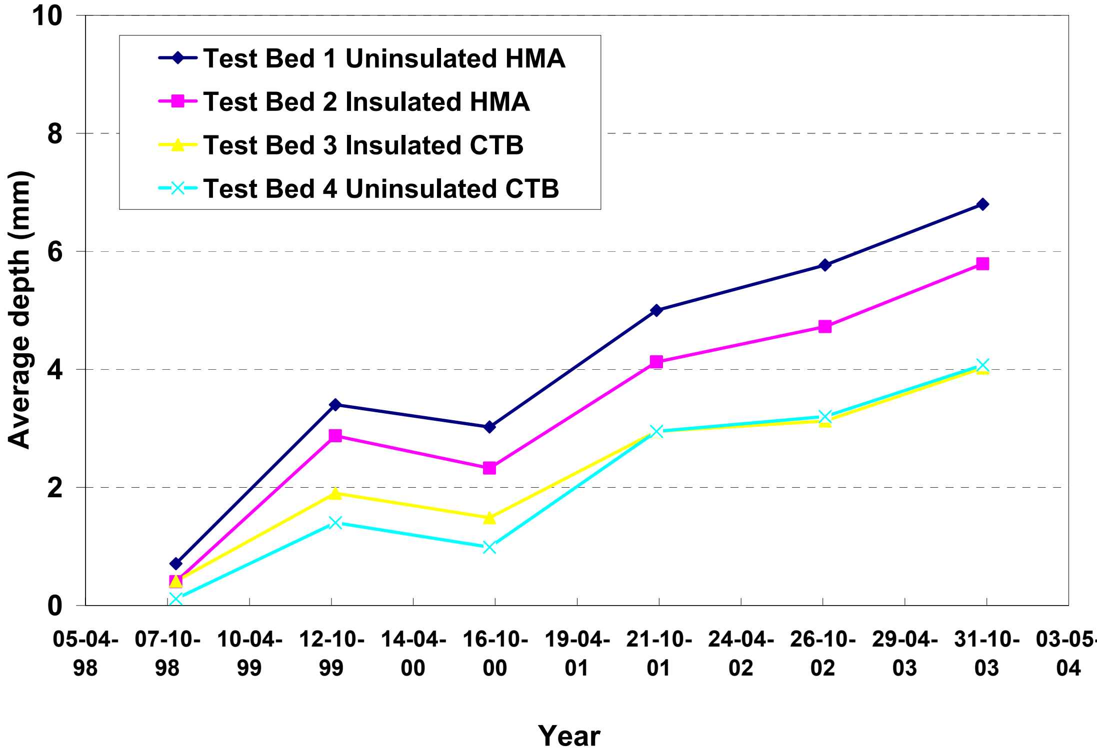


Figure 9 – Average rut depth trend (both directions)

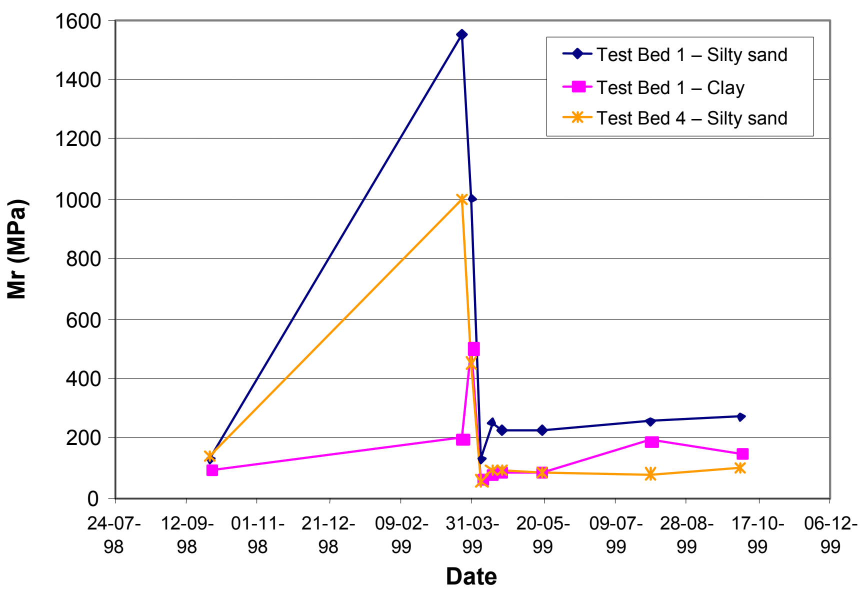


Figure 10 – Seasonal variation of the resilient modulus of the subgrade soil affected by frost for 1998-1999 (silty sand and clay for Test Bed 1 and silty sand for Test Bed 4)