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FIVE YEAR EXPERIENCE
ON
LOW TEMPERATURE PERFORMANCE
OF
RECYCLED HOT MIX

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FIVE YEAR EXPERIENCE

ON

LOW TEMPERATURE PERFORMANCE

OF

RECYCLED HOT MIX

by

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The opinions expressed in this report are those of the authors and do not necessarily reflect the official views or policies of the Ministry of Transportation of Ontario.

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FIVE YEAR EXPERIENCE ON LOW TEMPERATURE PERFORMANCE OF RECYCLED HOT MIX

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Abstract:

The Ontario Ministry of Transportation embarked on a recycled hot mix (RHM) program in 1979. Early observations of mixes placed in the first few years of the program indicated that RHM required improvement, particularly with respect to cracking. As a result, a long term monitoring program was planned. This paper presents the five-year monitoring results on five RHM sites, focusing mainly on the evaluation of their resistance to thermal cracking. The evaluation was undertaken as part of the long term monitoring program.

The resistance to low temperature cracking has been investigated in the laboratory, in terms of both limiting stiffness and fracture temperature criteria, and compared with field crack measurements. In the limiting stiffness approach, McLeod's tentative design guide was used to evaluate the resistance of the mixes to thermal cracking. In the case of fracture temperature criterion, the failure strains under uniaxial direct tension tests and the induced thermal strains were used to estimate the fracture temperature of recycled asphalt mixes. On the basis of both criteria, the test results confirm the common belief that recycled hot mixes are more susceptible to thermal cracking than non-recycled mixes. When the results of both criteria are compared with observed field data and recovered penetration and viscosity values, the fracture temperature approach appears to be more suitable than the stiffness criterion for evaluation of low temperature performance. Also, the good correlation between A.C. penetration results and field crack measurements confirms that mixes with higher penetrations have a better resistance to thermal cracking because of having lower fracture temperatures.

In order to reduce the susceptibility of pavements to thermal cracking, it is recommended that penetration values of RHM be improved by either reducing the recycling ratio and/or, using suitable grades of rejuvenating/virgin asphalt cement designed to the method developed by Tam [1]. Additionally, the fracture temperature method should be used to determine the cracking resistance property at the mix design stage to ensure the suitability of the mix for the expected winter temperatures.

Abrégé:

En 1979, le ministère des Transports de l'Ontario a mis sur pied un programme d'application d'enrobés à chaud recyclés. Au cours des premières années, l'étude de ces enrobés appliqués a démontré la nécessité d'apporter diverses modifications, notamment en ce qui a trait à leur résistance à la fissuration. Un programme de surveillance à long terme a donc été prévu. Le présent rapport porte sur les résultats de cinq années de surveillance de cinq tronçons recouverts d'enrobés à chaud recyclés et plus précisément sur l'évaluation de la résistance de ces enrobés à la fissuration thermique. Cette évaluation s'inscrivait dans le cadre du programme de surveillance à long terme.

La résistance à la fissuration à basse température a été étudiée en laboratoire selon les critères de la rigidité limite et de la température de cassure. Les résultats obtenus ont ensuite été comparés à la fissuration mesurée sur le terrain. Dans le cadre de l'étude de la rigidité limite, on a utilisé le guide de conception expérimentale de McLeod pour évaluer la résistance des enrobés à la fissuration thermique. En ce qui concerne la température de cassure, on a comparé les tensions de rupture exercées lors d'essais de traction uniaxiale directe aux tensions thermiques induites afin d'évaluer la température de cassure des enrobés bitumineux recyclés. Les résultats des essais effectués selon ces deux critères ont confirmé le sentiment général, à savoir que les enrobés à chaud recyclés sont plus susceptibles à la fissuration thermique que les enrobés non-recyclés. Si on compare les résultats obtenus selon les deux critères aux données recueillies sur le terrain et aux indices de pénétration et de viscosité, il semble préférable de se baser sur la température de cassure plutôt que sur la rigidité limite pour évaluer le rendement des enrobés à basse température. De plus, la corrélation entre les données de pénétration du ciment bitumineux et la fissuration enregistrée sur le terrain confirme que les enrobés dont l'indice de pénétration est plus élevé résistent mieux à la fissuration thermique du fait que leur température de cassure se situe à un niveau inférieur.

Afin d'accroître la résistance des revêtements à la fissuration thermique, on recommande d'améliorer l'indice de pénétration des enrobés à chaud recyclés soit en réduisant la proportion de matériaux recyclés, soit en utilisant des ciments bitumineux régénérés ou purs de bonne catégorie qui soient conformes aux normes de Tam [1]. On devrait également se baser sur la température de cassure pour déterminer la résistance des enrobés à la fissuration au moment de leur conception afin de s'assurer qu'ils seront adaptés aux conditions hivernales des régions auxquelles on les destine.

Keywords: Low temperature, thermal cracking, recycled asphalt, stiffness, fracture temperature, design guide, tension, penetration, viscosity

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1. INTRODUCTION

The Ontario Ministry of Transportation embarked on a hot mix recycling program in 1979. It was based on the findings of a task force set up to study the use of recycled hot mixes as alternatives for rehabilitating highway pavements. The task force also recommended an implementation package which called for an immediate commitment of substantial tonnages in the construction programs to encourage capital investment in equipment by paving contractors for the recycling of hot mixes.

By 1980, some half a million tonnes of recycled hot mix (RHM) were constructed, representing about 20 percent of the annual paving program [1]. Early observations of the performance of these mixes suggested that RHM needs improvement, particularly with respect to the resistance to thermal cracking. This created a need for an objective evaluation of RHM, both in the laboratory as well as long term monitoring in the field. An experimental program was therefore initiated, including a series of laboratory tests and field crack mapping data to evaluate the properties of RHM. This paper presents the five-year field monitoring results and findings on the part of study on low temperature performance of RHM placed during the first few years of the hot mix recycling program.

2. SCOPE OF EVALUATION

The primary objective of the study is to investigate the relative resistance of recycled hot mixes to thermal cracking, as compared to conventional hot mix. However, having reviewed available evaluation techniques, it was found necessary also to examine and decide on the most appropriate methodology for evaluating the low temperature susceptibility of hot mix in the future. In more specific terms, the objectives can be stated as follows:

(1) to review and compare existing methods for low temperature evaluation of hot mix;

(2) to design an experimental program to evaluate the criteria for the methods selected in step 1;

(3) to analyse the test results and field measurements, and compare the low temperature performance of various RHM with a conventional hot mix;

(4) to determine if laboratory mixed samples can be used for low temperature evaluation instead of plant mixed samples;

(5) to identify the best method for future low temperature evaluations of hot mix based on laboratory and field results;

(6) to suggest improvement, if any, for future mix design procedures.

3. EVALUATION METHOD

A number of procedures have been developed for evaluating low temperature susceptibility of bituminous materials and they can be categorized as follows:

(a) use of asphalt binder rheology,
(b) use of limiting stiffness,
(c) predicting fracture temperature, and
(d) estimating cracking frequency.

Asphalt binder rheology has been considered a controlling factor in cold temperature cracking. However, its use is not applicable to this case because it deals with only the binder and not the overall performance of asphaltic mixes. The approach of limiting stiffnesses has evolved, as a result of field observations and much of the laboratory work [2 & 3]. The method involves the determination of mix stiffness and compares such values with an established design guide [4]. If the estimated value exceeds the design value, cracking is likely to occur.

The prediction of fracture temperature is based on the thermally induced stresses or strains in the bituminous layer. When the induced stress or strain, due to temperature drop, exceeds the failure stress or strain, cracking is expected to occur. The corresponding temperature is called the fracture temperature. The higher is the fracture temperature of a material, the lower its resistance to thermal cracking. In a design situation, if the estimated fracture temperature is less than the winter design temperature [5], cracking is expected to occur and the design should be rejected or modified.

The approach of estimating cracking frequency developed by Haas [5] is based on the stiffness of asphalt cement, and not the asphaltic mix. As such, this method is not considered in this investigation.

In summary, the method of evaluation for this study is confined to two criteria: limiting stiffness and fracture temperature.

4. LABORATORY EXPERIMENT

A laboratory test program was designed specifically to investigate the potential of different asphaltic mixes to resist thermal cracking, using limiting stiffness and fracture temperature criteria. The program consists of a series of tests, performed on a conventional hot mix (HL 4, similar to an Asphalt Institute IV B dense graded mix) and different RHM specimens, to establish performance indicators for the mixes. The type of tests included in the program are:

(a) direct tension tests at various temperatures, and
(b) thermal contraction tests.

A detailed description of theories, applications and interpretation of results of the direct tension test can be found in reference [6].

4.1 Material Selection

Materials were selected from five recycling contracts (81-74, 81-224, 82-04, 82-19 and 83-43), which cover different regions, virgin asphalt cements (A.C.) and recycling ratios. The locations and traffic volumes of different RHM sites selected are shown in Table 1. The aggregate types and asphalt binder grades used are given in Tables 2 and 3, respectively.

For the RHM, specimens were produced from plant mixes and individual mix components in the laboratory. However, for Contracts 81-74 and 82-04, the A.C. used for making laboratory mixes was different from that for the plant mixes, because of non-availability of the binders as used in the field. Penetration grade asphalt cements, ranging from 85/100 to 300/400, were used for the RHM. Also, for Contract 83-43, there was only plant mix available for making test specimens.

A crushed limestone aggregate and an 85/100 penetration grade A.C. were used for making conventional mix (HL 4) specimens.

4.2 Mix Designs

The RHM mix designs cover different recycling ratios, ranging from 25/75 to 70/30. They were designed to meet the usual Ministry specification requirements. Except for Contract 83-43 where the mix was designed using an HL 1 RAP material from a surface course of a freeway, the others were HL 4 mixes for secondary highways. The HL 1 RAP material consisted of a good quality steel slag coarse aggregate and natural sand fine aggregate. The mix design properties are shown in Table 3 and mix gradations in Table 4.

Due to the use of various recycling ratios and RAP with low recovered penetrations, virgin A.C. of different penetrations were used to bring the overall penetration of the RHM to desirable levels [1], which are normally obtainable for virgin mixes.

4.3 Sample Preparation

Test samples were prepared according to procedures given in ASTM D3202, using the California kneading compactor. Compacted beam specimens of length 381 mm, depth 88 mm and width 82 mm were saw cut into three shorter sections of 100 mm in length and 76 X 76 mm^2 in cross-section. For the direct tension test, each specimen was glued to steel end-plates with epoxy, and allowed to cure overnight before temperature conditioning and testing. For thermal contraction measurements, the specimens were used as cut without further additional treatment.

4.4 Direct Tension Test

Direct tension tests were performed, using a MTS servo-hydraulic machine at 21°C, -5°C and -35°C, to determine the tensile strengths, strains, and stiffnesses of different mixes. Duplicate tests were performed for each of the test temperatures (i.e. 21°C, -5°C and -35°C) and sample preparation conditions (i.e. laboratory and plant mixing). The specimen was pulled at a constant rate of extension (3 x 10^{-2} mm/min for tests at -5°C and -35°C, and 5 mm/min at 21°C) until failure occurred. The stress, strain and stiffness at failure were determined (Table 5). For testing at temperatures -5°C and -35°C, the specimens were pre-cooled overnight prior to testing. An environmental chamber was used to maintain the low temperatures of -5°C and -35°C.

4.5 Thermal Contraction

Determination of thermal contraction is an integral part of the analysis of the thermal behaviour of asphalt mixes. Thermal contraction was used to estimate the induced strain due to thermal shrinkage under restraint condition. Set points were selected on each sample and the lengths were measured using a digital calliper before and after cooling the specimens for 24 hours at each temperature. Readings were taken during both the cooling and heating cycle for comparison. Thermally induced strains at various temperatures are calculated for all mixes by dividing the actual contraction of each specimen by its original length (Tables 6 and 7).

5. RESULTS

Analysis of the low temperature cracking properties of the mixes is made in terms of both limiting mix stiffness and pavement fracture temperature criteria. Also, the results obtained from plant mixes are compared with those from laboratory mixes. All of these results are then, in turn, compared with the HL 4 mix and with field data and expected trends, using the recovered penetration and viscosity values of the asphalt cements used.

5.1 Limiting Mix Stiffness Criterion

In general, the mix stiffness increases with decrease in temperature (Figs. 1a to 1c). McLeod's tentative design guide [4] has been used for the limiting stiffness analysis. The stiffnesses of RHM at the time of construction and after five years of service and the conventional mix at various temperatures are compared with McLeod's maximum criterion. It appears that none of the mixes has resistance to cracking at low temperature (Figs. 1a, 1b and 1c). However, it is noted that McLeod's values for limiting stiffness, according to the Asphalt Institute Research Report [5], may be too conservative because of its dependency on the PVN values, which tend to give lower stiffnesses than the actual measured values. Additional safety factors could have been built into such procedures to produce these limiting values. This assumption is confirmed by comparing the McLeod's maximum stiffness values at which cracking is expected to occur with the measured values at the fracture temperatures. Results show that a range of factors of about 2 at -23°C and 7 at -13°C existed when dividing the stiffness values at fracture temperatures by McLeod's values (Table 5). The stiffness values at fracture temperature were obtained from the graphs in Figs. 1a to 1c at the fracture temperatures shown in Figs. 4a, 4b and 4c for the different mixes.

5.1.1 *Relationship of Plant and Laboratory Mix Stiffnesses*

One of the objectives of this study has been to determine if laboratory mixed samples can be used, instead of plant mixed samples, for evaluation of the performance of RHM or other mixes at low temperature. This point is addressed by comparing the stiffnesses of mixes produced in the plant and in the laboratory (Fig. 2). Laboratory mixed specimens exhibit higher stiffness values (ranging from 21 percent to 54 percent), at warmer temperatures i.e. 21°C, than the plant mixed specimens, and lower stiffness values (ranging from 38 percent to 142 percent, except for Contract 81-74), at very low temperatures i.e. -35°C. From these results, it is clear that the stiffness value at any given temperature is not sufficient to adequately evaluate the behaviour of asphaltic mixes at low temperatures. The percentage difference between laboratory and plant mix stiffnesses appears significant, averaging to about 30 percent for both ends of the temperature range (Table 5), when the stiffness value for the mix from Contract 82-04 was excluded. However, a plot of the logarithmic values (Fig. 3) shows excellent correlation (r^2 = 0.976) between the plant and laboratory mix stiffnesses. The difference in stiffness, between the plant and laboratory mixed specimens, may be due to the different degrees of binder hardening as a result of the different mixing and production procedures used for preparing the mixes.

Based on the good correlation between the stiffness values obtained from the laboratory and plant mixes, future low temperature stiffness evaluation of RHM can reliably use samples that are prepared in the laboratory.

5.1.2 *Comparison of Virgin Mix and RHM*

The RHM from all contracts show higher stiffness values than the HL 4 mix. This confirms the general perception that virgin mixes will perform better than RHM with respect to thermal cracking.

5.2 Fracture Temperature Criterion

Fracture temperature (F.T.) of a pavement is defined as the temperature at which the pavement cracks due to shrinkage. It could be estimated by plotting the failure strains, determined from direct tension tests and the induced thermal strains (examples given in Tables 6 and 7) of different hot mixes against decreasing temperature, as shown in Figs. 4a, 4b and 4c. The temperature at which the induced thermal strain exceeds the failure strain corresponds to the F.T. of the mix.

5.2.1 *Relationship of Plant and Laboratory Mix Fracture Temperature*

Laboratory mixed samples generally have higher F.T. than plant mixes (Fig. 5). This indicates that laboratory samples have a lower resistance to thermal cracking than plant samples. The trend is based on the results of the three Contracts shown in Fig. 5, and excluding the value for Contract 82-04 because of its much harder binder properties (Fig. 6), as will be discussed later. As mentioned previously under 5.1.1, the difference could be associated with the different mix production procedures. However, as can be seen in Fig. 5, there is a clear relationship between the F.T. for the plant and laboratory mixes. In this respect, it is possible to use laboratory mix to evaluate field performance, once the trend is established or substantiated by more data.

The lower resistance to thermal cracking of the laboratory mixes is only in agreement with the results of stiffness analysis at 21°C, and not the -35°C values (Table 5). Since the intent of this evaluation is to determine the low temperature thermal cracking properties of asphalt mixes, stiffness measurement at -35°C is considered more appropriately related to the F.T. results than those of the 21°C. Also, as stiffness value at any given temperature is not sufficient to evaluate low temperature performance, and as F.T. is a more direct measurement of the fracture property of a mixture, F.T. approach should therefore be more reliable for predicting low temperature performance.

5.2.2 *Comparison of Virgin Mix and RHM*

Except for Contract 82-19, all other mixes had F.T. above the -23°C F.T. of the virgin HL 4 mix (Fig. 4b). This again confirms the previous findings, based on the stiffness modulus, and the general belief that RHM is more susceptible to thermal cracking than virgin mixes.

5.3 Effect of Aging on Stiffness and Fracture Temperature

As expected, after five years of aging, the resistance to low temperature cracking of pavements has reduced on the basis of both F.T. (Figs. 4a and 4c) and stiffness at F.T. criteria. In addition to an average increase of 10°C in the F.T., there is a corresponding increase in stiffness averaging to 1700 MPa over the five year period (Fig. 7).

5.4 Effect of Binder Properties on Stiffness and Fracture Temperature

As fracture characteristics of asphalt mixes are generally believed to be more affected by the properties of the binder used, the relationships of the asphalt cement penetration and viscosity and mix stiffness and F.T. are therefore examined.

Figs. 7a and 7b show that mix stiffnesses at F.T. bear little relationship with the recovered penetration, but have a slight trend with the viscosity values of RHM. This confirms the common belief that the stiffness of a mix is more related to the viscosity than the penetration of the binder used.

However, a better relationship is obtained when mix F.T. is correlated with the results of recovered asphalt cement penetration and viscosity of the RHM (Figs. 8a & 8b). On the basis of these data, mixes with recovered penetration values of about 75, or viscosity of about 350 mm^2/s at the time of construction, can be expected to be able to resist cracking to as low a temperature as -30°C. This is a significant finding in that it confirms the empirical rule or common belief that recovered penetration values of about 60 is sufficient for Southern Ontario, and values greater than 70 are necessary for Northern Ontario, for resisting low temperature pavement cracking. Also, this finding further emphasizes the importance of designing all asphalt mixes to meet penetration requirements. At the present state of the technology, this can be achieved either by using historic data on the percent retained penetration values after the TFOT (or RTFT) for conventional mixes or the method developed by Tam as described in [1] for RHM.

5.5 Field Observation

Field condition surveys were carried out periodically including crack mapping of different types of crack as shown in Table 8. The crack surveys data, in terms of metres of cracks per lane kilometre, is compared with the laboratory test results. It is noted that the cracks measured might be considered exclusively due to the thermal effect rather than to loading because of the structural soundness of the monitored sites. All of these sites were constructed with good base supports (400 - 500 mm granular), and thick asphalt layers (90 - 150 mm). As well, the traffic volumes were low (700 - 1500 AADT, Table 1), except for Contract 83-43 which carried 35,780 AADT (1987) but was a full depth asphalt pavement (200 mm).

After five years of service, contracts 82-19, 81-224 and 81-74 have total pavement crack lengths between 230 and 800 m as against to the lengths of about 3,800 m for Contracts 82-04 and 83-43. This indicates that the RHM from the former three Contracts are less susceptible to cracking than from Contracts 82-04 and 83-43. This is in apparent agreement with the findings so far obtained, particularly in respect to the higher F.T. and lower recovered penetration values obtained for the latter two contracts as shown in Fig. 6a. In general, there is an average decrease in penetration of 18 units and an increase in viscosity of 230 mm^2/s over the five year period.

When the relationships of metres of cracks per lane kilometre, measured at different age of the pavements, and their respective F.T. and stiffness are being examined, a good correlation of $r^2 = 0.913$ and 0.897 for F.T. (Fig. 9a) and a poor correlation of $r^2 = 0.484$ and 0.065 for stiffness (Fig. 9b) are obtained. These relationships confirm the better use of F.T. than stiffness criteria for evaluating low temperature cracking. Also, excellent relationships between crack lengths and recovered RHM penetration and viscosity (Figs. 10a and 10b) are obtained. These relationships are significant in demonstrating that:

(a) The use of F.T. to predict future crack lengths of aged pavements is possible, pending on further substantiation by more data.

(b) The common approach of using penetration values at 25°C, and viscosities at 135°C, for selecting the A.C. grades in mix designs for low temperature performance appears to be valid, provided other requirements are being met.

Following a similar plot as Fig. 10, using the recovered penetration values at construction, instead of at different time, Fig. 11 shows that there was a bigger increase in crack lengths for the first year than the subsequent years. When a mix has penetration value of below 20, a substantial crack length of 20,000 m per kilometre of road can be expected within the first year, i.e. at the state of complete disintegration of the pavement.

5.6 Performance Rating of Mixtures

For the purpose of evaluating the relative resistance of the mixes to thermal cracking, a rating system of 1 to 5, (corresponds to very good to poor resistance) is used. The mixes are rated by their stiffness values, fracture temperatures and field data (Table 9). Contract 82-19 and conventional HL 4 mixes show consistently better ratings than the others, except when they were rated under the stiffness criteria at -35°C. The other mixes from Contracts 82-04 and 83-43 were rated poor for causes as discussed in 5.3. The rating is consistent with the field results shown in 5.4. The RHM (Contracts 82-19, 81-224, 81-74) that performed well are those mixes with recycling ratios at or below 60/40, 50 percent or more and of 300/400 penetration grade virgin asphalt cement, and recovered penetration values at the time of construction similar to conventional mixes, such as the HL 4 mix used in this study.

6. PREDICTION METHODS FOR LOW TEMPERATURE PERFORMANCE

The two approaches, McLeod's limiting stiffness criteria and F.T., are further examined below in terms of both field data and material standards e.g. penetration values, as to their suitability for predicting low temperature performance of hot mixes.

(a) Appropriateness of limiting stiffness -- previous discussion (in 5.1.1 and 5.2.1) stated that McLeod's values are considered conservative and inadequate for predicting low temperature behaviours of hot mixes. It is evident in Fig. 12 that McLeod's limit is well below the fracture stiffness values obtained for this study. McLeod's values may not be just conservative, because a family of limits could exist for different types of mixes e.g. rich, medium and harsh/dry mixes. Possibly, McLeod's data was for the early richer and finer asphaltic mixes of higher binder content type (e.g. 6 percent). The results of the "medium" mix type, represented by the three good mixes used in this study, are plotted as a solid line in Fig. 12. A good correlation with $r^2 = 0.997$ is obtained. The dotted line is for the "dry" and aged mix type, similar to the mixes used in Contracts 82-04 and 83-43. If this argument holds true, the limiting stiffness method will have to be further examined and modified accordingly before it can be used with confidence.

(b) Appropriateness of fracture temperature -- the findings so far obtained reveal that the F.T. approach is more suitable and reliable for evaluating low temperature performance of hot mixes, for the following reasons:

* It has been substantiated by the good correlation with field data.
* It correlates well with recovered penetration values and follows the expected trends of behaviours i.e. higher penetration corresponds to lower fracture temperature, and increase in F.T. with aging of pavements.
* It is a direct measurement and can be compared with expected local/winter temperature of a given pavement, to ensure that a mix to be constructed has adequate low F.T. properties.

In general, despite the common practice of using factors such as penetration (including penetration index and pen-visc number), resilient modulus and viscosity gradients etc. to predict low temperature performance of asphaltic mixes, it appears that the fracture temperature is the better method in the final analysis. Other methods become less significant because of the indirect nature of the tests, although these tests could be of value in assisting in screening potential bad actors.

7. CONCLUSIONS

This study reveals the following findings. They are considered significant because they have contributed to clear up some of the long standing doubt on the correctness of past practices, and identify the use of fracture temperature (F.T.) as a preferred and direct method for testing of low temperature performance of asphalt mixes.

Materials Performance

(a) The findings confirmed the general belief that recycled hot mixes are more susceptible to thermal cracking than conventional hot mixes.

(b) Based on the fracture temperature results, recycled hot mixes from Contracts 81-74, 82-19 and 82-224 have better resistance to cracking than those from Contracts 82-04 and 83-43.

(c) Both field data and laboratory results reveal that hot mixes recycled at low recycling ratios (e.g. 30/70 as in Contract 82-19), or using high penetration virgin asphalt cement (e.g. 300/400 as in Contracts 81-74 and 81-224), have better performance than those with high recycling ratios (e.g. 70/30 as in Contract 82-04) or using low penetration virgin asphalt cement (e.g. 85/100 as in Contract 83-43).

(d) Based on the RHM studied, an average increase in F.T. of 10°C and a corresponding increase in stiffness of 1700 MPa can be expected for a pavement after five years of service. Also, a reduction in penetration of about 16 units and an increase in viscosity value of about 230 mm^2/s have been observed.

(e) The findings (i) reveal that an asphalt cement with a recovered penetration of 75 and a viscosity of 350mm^2/s can resist cracking to - 30°C (ii) confirm that the use of higher penetration grade asphalt cement (e.g. 150/200) in a standard mix in Northern Ontario is appropriate, because it has an expected F.T. of below -35°C (iii) support the common belief that higher penetration mixes tend to have lower fracture temperatures or better resistance to low temperature cracking.

Evaluation Methods

(f) Fracture temperature method is considered more suitable than limiting stiffness criteria for evaluating the potential of mixes to resist low temperature cracking.

(g) Fracture temperature ratings of mixes used in this study correlate well with ratings based on field data.

(h) Fracture temperatures also correlate well with recovered penetration values which are supported by the common experience that higher penetration mixes tend to have better resistance to low temperature cracking and vice versa.

Laboratory Sample for Predicting Low Temperature Performance

(i) It appears from the limited data of this study that laboratory manufactured samples could be used for predicting low temperature performance.

8. RECOMMENDATIONS

The following recommendations are made, to minimize low temperature cracking and improve the accuracy of predicting fracture temperature:

(a) limit recycling ratios to a maximum of 50/50;

(b) select an appropriate virgin asphalt cement to produce desirable final/recovered mix penetrations using the method developed by Tam as described in [1];

(c) use fracture temperature method for mix evaluation to ensure compliance with expected winter temperature;

(d) obtain more data to support the use of laboratory samples for prediction/evaluation of low temperature performance in the field, for both RHM and conventional mixes.

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Table 1: Location and Traffic Volume (AADT) for Different RHM Monitoring Sections

CONTRACT	REGION	HWY	LOCATION	AADT 1981	AADT 1987
81-74	EASTERN	60	35km E. of Renfrew	1500	1500
81-224	NORTH WESTERN	11	30km E. of Hwy633	600	680
82-04	SOUTH WESTERN	21	5km S. of Thamesville	1900	2100
82-19	CENTRAL	649	10km N. of Bobcaygeon	1000	1050
83-43	CENTRAL	QEW	Niag. Falls	26,200	35,780

Table 2: Aggregate Description for Various Mixes

Mix Identification	Coarse Aggregate	Fine Aggregate
81-74	Crushed gravel containing Palaeozoic carbonates and marble granite	Silica rich natural sand
81-224	Crushed igneous gravel with mainly granite and granite gneiss	Silica rich natural sand
82-04	Crushed carbonate gravel with < 10% granite and gneiss	Silica rich natural sand
82-19	Crushed limestone gravel	Carbonate & silicate natural sand
83-43	Steel slag (DOFASCO)	Steel slag (DOFASCO)
HL-4	95% carbonate rock, 5% granite	95% carbonate rock, 5% granite

Table 3: Mix Design Properties of Recycled Hot Mixes and HL4 Mix

CONTRACT NUMBER (Mix Type)	RECYCLING RATIO	ASPHALT CEMENT, % ADDED	ASPHALT CEMENT, % TOTAL	ASPHALT CEMENT, % PEN	Stability N	Flow .25mm	VMA %	Ret., % 4.75mm	Air Voids %
81-224 (HL4)	60/40	2.7	5.2	300/400	10,000 (16,887)	9.2 (11.4)	14.6 (14.6)	40	2.8 (1.4)
82-19 (HL4)	30/70	4.5	5.3	150/200	10,000 (12,080)	13.5 (14.2)	12.5 (14.3)	45	1.6 (1.0)
81-74 (HL4)	50/50	2.3*	5.0	300/400	16,760 (14,469)	12.5 (12.1)	N/A (14.8)	45	2.8 (2.8)
82-04 (HL4)	70/30	1.7	5.3	150/200	16,280 (17,658)	13.8 (13.5)	15.6 (15.3)	36	3.0 (1.9)
83-43 (HL1)	25/75	3.8	5.3	85/100	23,000 (24,042)	18.1 (17.4)	11.8 (9.5)	45	2.7 (2.6)
(HL4)	-	-	5.3	85/100	13600	10.4	13.8	45	4.0

** 0.5% antistripping agent added*

Plant check results are given in parentheses

Table 4: Aggregate Gradations for HL4 and RHM Mixes

Mix		% Retained											
		Sieve size, mm								Sieve size, µm			
		19	16	13.2	9.5	6.7	4.75	2.36	1.18	600	300	150	75
HL4	CA			23.2	68.5	89.0	98.1	98.3	98.5	98.7	98.8	99.0	99.2
	FA						1.0	20.1	39.0	58.6	82.4	92.4	96.7
	Final			10.5	31.0	40.3	45.0	55.5	65.9	76.7	89.8	95.4	97.8
81-74	RAP			3.2	13.4	26.0	36.8	50.7	60.9	71.5	84.1	91.0	94.8
	CA		4.3	20.4	63.7	88.3	92.7	93.6	94.5	95.4	96.2	97.1	98.0
	FA						0.4	1.4	5.3	21.0	60.6	88.8	95.9
	Final		1.3	7.5	25.1	38.4	45.0	52.2	58.3	67.2	82.4	92.3	96.0
81-224	RAP		1.3	5.8	24.0	38.9	49.0	62.4	72.9	81.2	87.6	91.7	94.9
	CA		2.8	17.2	55.7	81.6	94.6	95.2	95.7	96.3	96.9	97.4	98.0
	FA					1.0	9.1	29.4	51.3	72.2	87.8	95.1	97.9
	BLSA	0.5	1.0	1.6	3.6	5.7	8.3	17.1	32.2	55.8	85.3	97.5	99.4
	Final		1.0	4.9	19.6	31.2	40.0	52.6	94.9	77.0	88.1	93.7	96.4
82-04	RAP	2.0	4.0	7.0	18.0	33.0	44.2	57.0	64.0	71.0	84.0	93.0	95.5
	CA		0.3	8.5	59.9	88.7	98.1	98.3	98.6	98.8	98.0	89.3	99.5
	FA				0.5	1.0	2.0	4.1	8.0	14.5	33.9	81.7	94.2
	Final	1.4	2.8	5.3	15.6	27.5	36.0	45.4	51.2	57.8	71.8	90.4	95.4
82-19	RAP	1.3	6.7	10.0	23.0	34.7	45.0	59.7	70.3	79.0	87.0	93.0	96.0
	CA		3.5	19.5	63.4	88.5	97.6	97.9	98.2	98.5	98.8	99.1	99.0
	FA						1.9	16.4	35.1	51.5	70.2	83.1	91.1
	BLSA				1.4	2.9	5.3	12.3	24.1	39.8	65.6	88.5	96.1
	Final	0.4	3.1	9.0	26.8	38.3	45.0	53.7	62.9	71.9	83.1	92.1	96.2
83-43	RAP			1.0	8.0	22.0	32.0	46.0	58.0	68.0	80.0	89.0	92.8
	CA			2.0	34.6	69.7	94.5	95.2	95.9	96.7	97.4	98.1	98.8
	FA							39.1	62.4	75.4	85.0	91.1	95.2
	Final			1.0	15.6	32.8	45.0	62.9	74.6	92.2	88.7	93.4	96.0

Table 5: Stiffness Modulus Values (MPa) of Plant and Laboratory Mixes

Mix	@ Test Temperature			% Difference, Plant/Lab		@ Fracture Temperature		Estimated / Mcleod's
	21°C	-5°C	-35°C	21°C	-35°C	Estimated from graph (Figs.1a&b)	McLeod's Maximum	
HL4L	36	460.0	20618	-	-	5000	2760	1.8
81-74P	268.0	797.0	24635	-21	-30	3800	1800	2.1
81-74L	338.5	1275.5	35280			2800	790	3.5
81-224P	157.5	637.2	34341	-35	38	6100	2760	2.2
81-224L	243.0	938.0	24920			2900	1100	2.6
82-04P	754.0	2761.5	53940	-54	142	5400	790	6.8
82-04L	1631.5	3258.5	22231			5400	790	6.8
82-19P	148.4	480.6	38645	-37	20	9000	3200	2.8
82-19L	237.0	923.0	32315			7800	3050	2.6
83-43P	1151.0	3032.0	24185	-	-	3600	-	-

Table 6: Thermally Induced Strains ($\times 10^{-3}$ mm/mm) for Plant Mixes

Temperature °C	HL4	Contract				
		81-74	81-224	82-04	82-19	83-43
-38	1.22	0.986	0.7	0.91	0.71	1.0
-29	1.12	0.796	0.7	0.815	0.71	0.8
-7	0.51	0.696	0.335	0.509	0.339	0.635
4	0.20	0.497	0.335	0.3	0.271	0.440

Table 7: Thermally Induced Strains (x10 $^{-3}$mm/mm) for Laboratory Mixes

Temperature (°C)	Contract			
	81-74	81-224	82-04	82-19
-38	1.479	1.49	0.689	0.996
-8	-	-	0.49	0.49
-5	.39	.49	-	-

Table 8: Crack Map Data After 3 and 5 Years of Service

Years After Construction	Contract	Cracks, m/km lane					
		Type A	Type B	Type C	Type D	Type T	Total
3	81-74	53	249	97	18	0	417
	81-224	30	168	127	0	0	325
	82-04	198	2068	492	151	1000	3909
	82-19	65	54	0	0	0	119
	83-43	190	108	5	0	2849	3152
5	81-74	267	418	117	0	0	802
	81-224	30	181	180	0	0	391
	82-04	198	2163	587	151	1000	4099
	82-19	113	121	0	0	0	234
	83-43	111	334	0	0	3014	3459

Note: *A = Hairline crack*
B < 5mm
C =5-10mm
D =10-20mm
T = Treated

Table 9: Cracking Resistance Rating of Different Mixes
Based on Fracture Temperature, Stiffness at -35°C and the Field Data

Cracking Resistance Ranking	Mix (by Contract Number)				
	Fracture Temperature		Stiffness at -35°C		Field Data
	Plant	Lab	Plant	Lab	
1 (Very High)	82-19	HL4 82-19	81-74	HL4 82-04	82-19
2 (High)	81-224	81-224	81-224	81-224	81-224
3 (Medium)	81-74	81-74 82-04	82-19	82-19	81-74
4 (Poor)	82-04 83-43	-	82-04	81-74	82-04 83-43

Fig. 1: Fracture Stiffness and Temperature Gradients

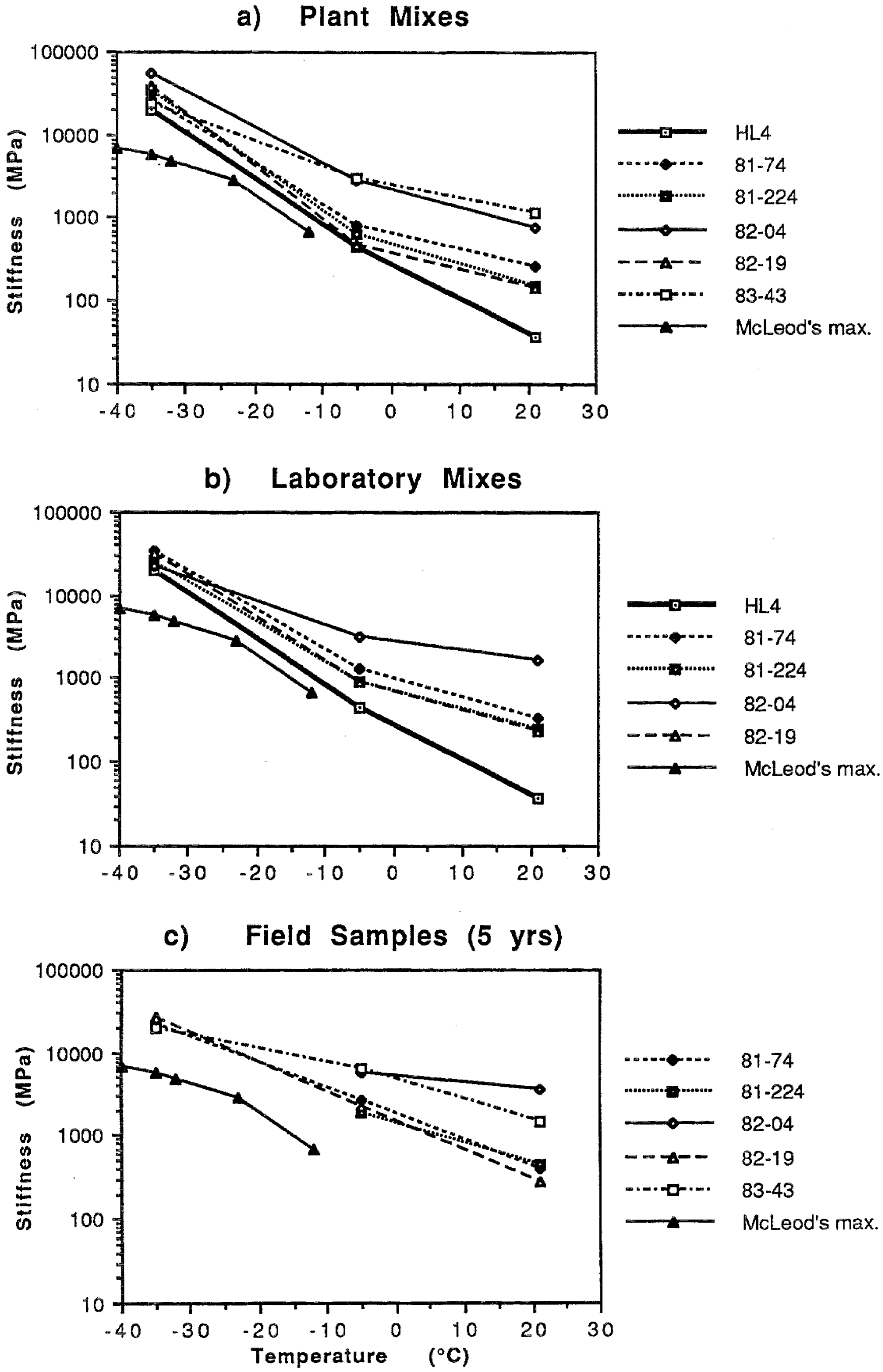


Fig. 2: Comparison of Field, Plant and Laboratory Mix Stiffnesses at Different Temperatures

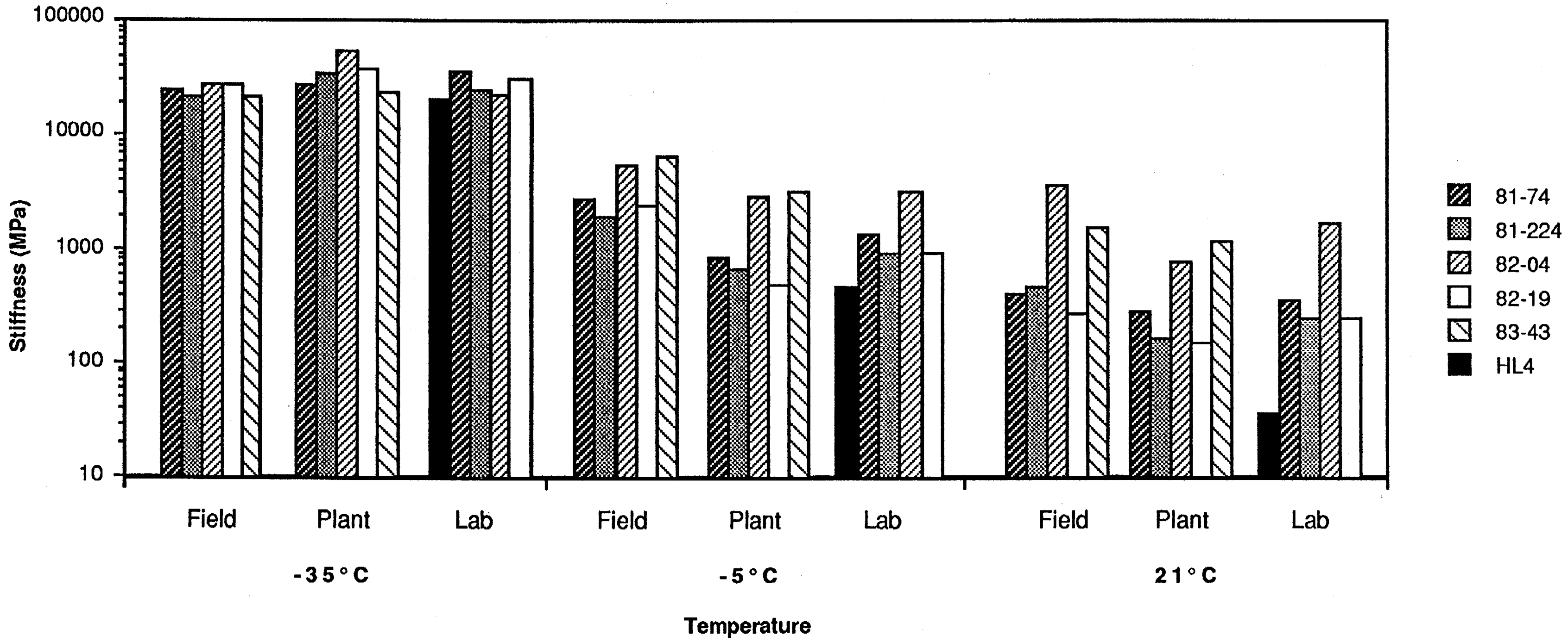


Fig. 3: Relationship of Laboratory and Plant Mix Stiffnesses at -35°C, -5°C, and 21°C

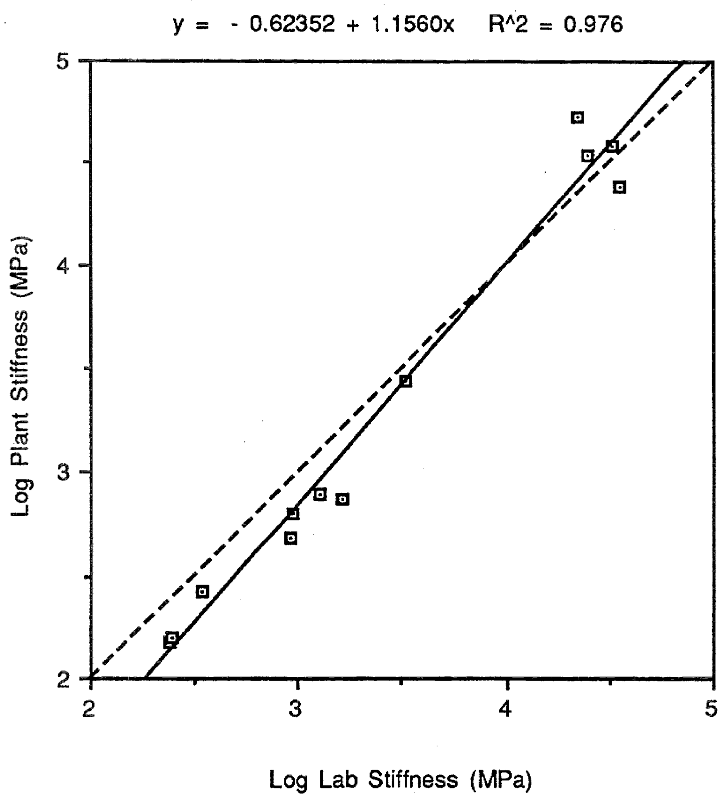


Fig. 4a: Estimation of Fracture Temperature for Plant Mixes

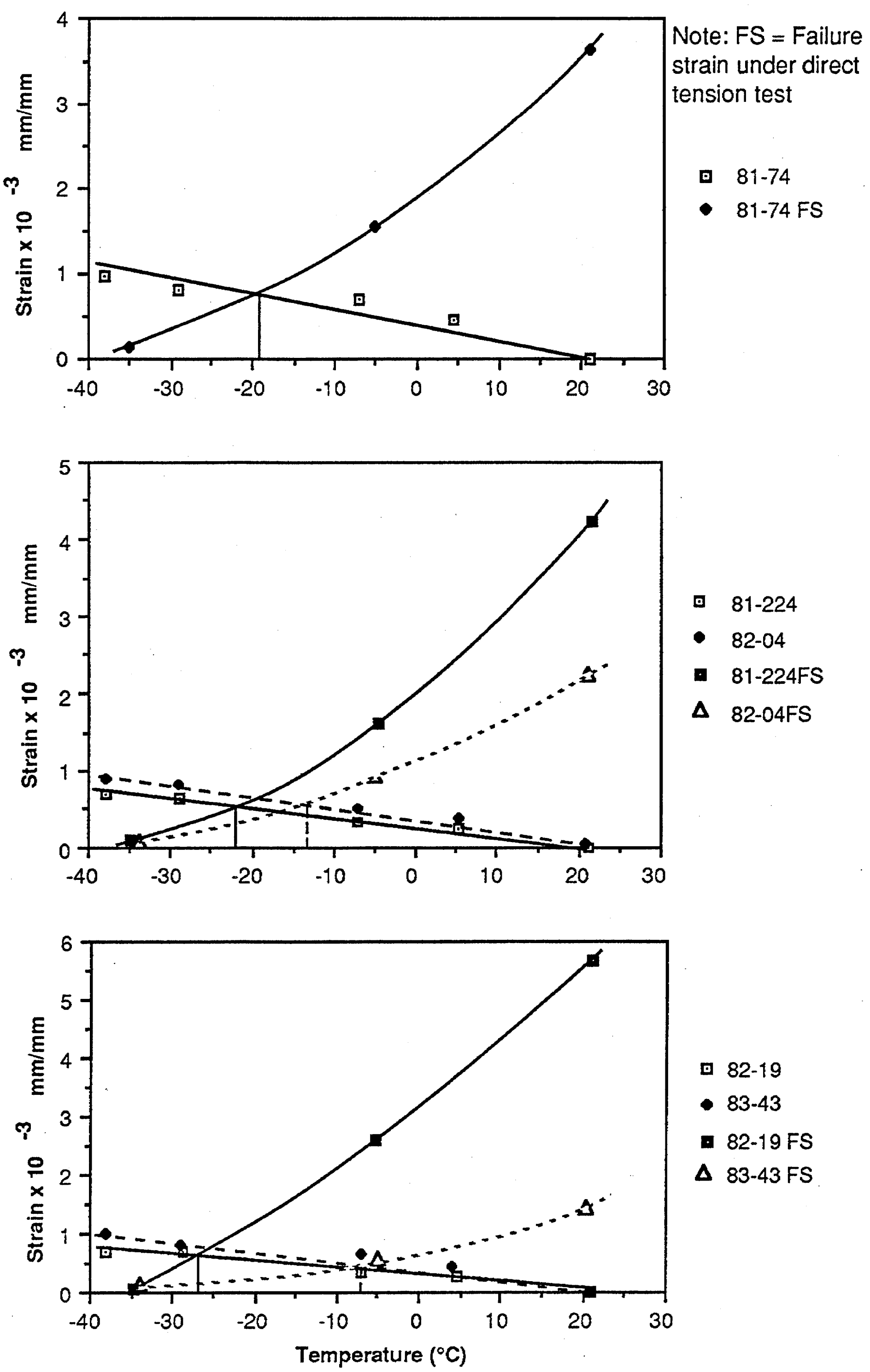


Fig. 4b: Estimation of Fracture Temperature for Laboratory Mixes

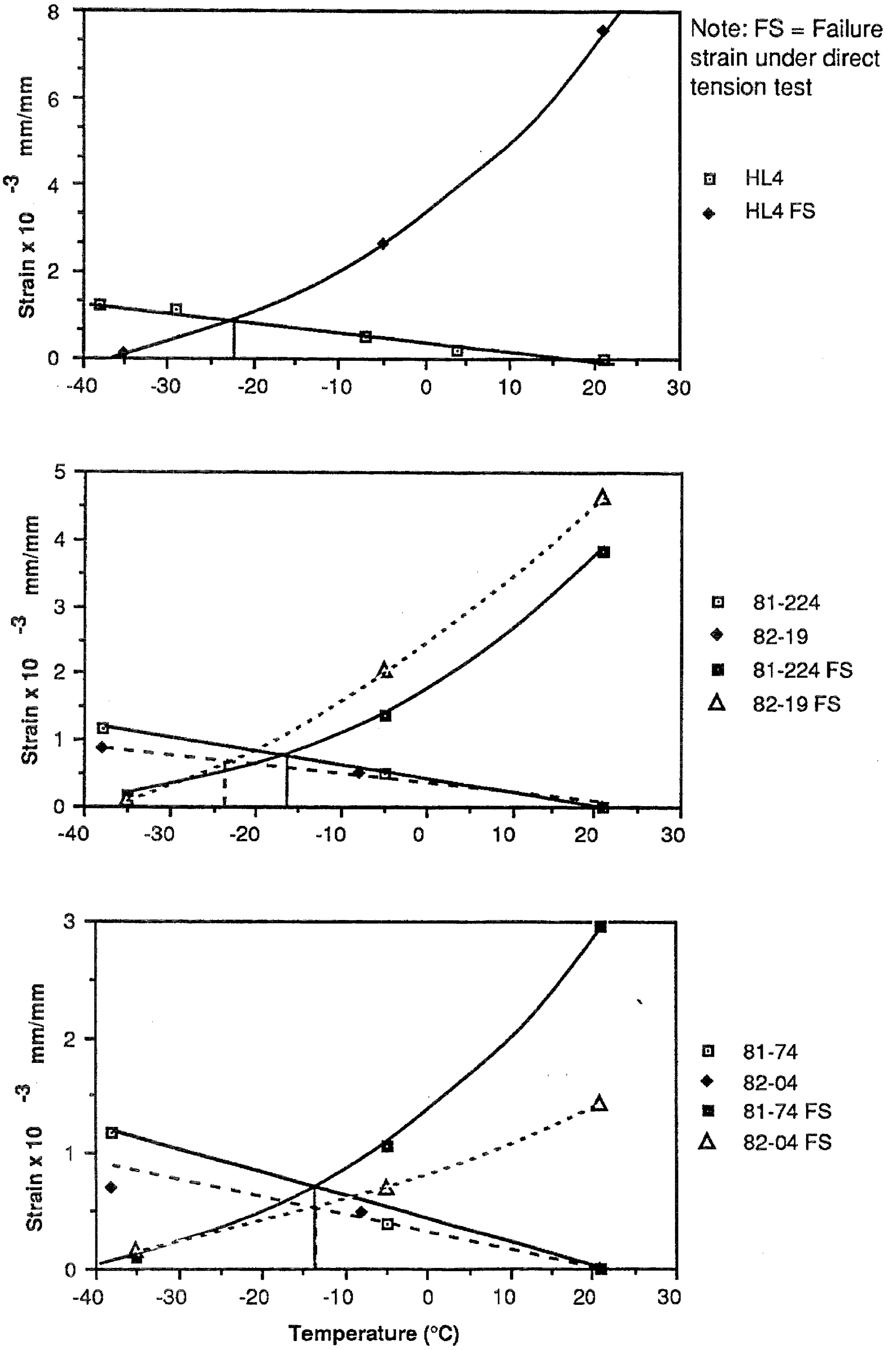


Fig. 4c: Estimation of Fracture Temperature for Field Mixes at 5 Years

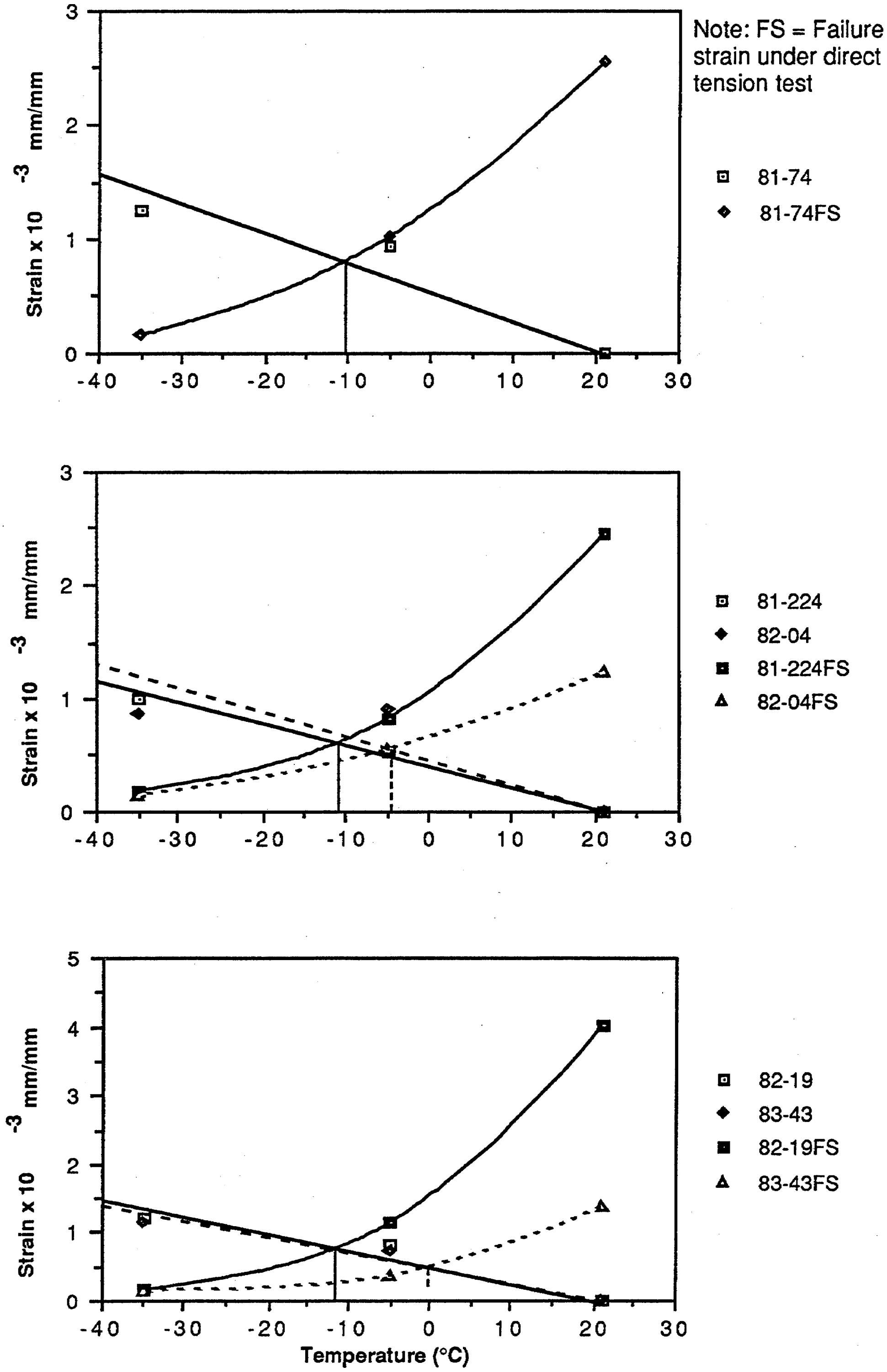


Fig. 5: Comparison of Plant and Lab Mix Fracture Temperatures

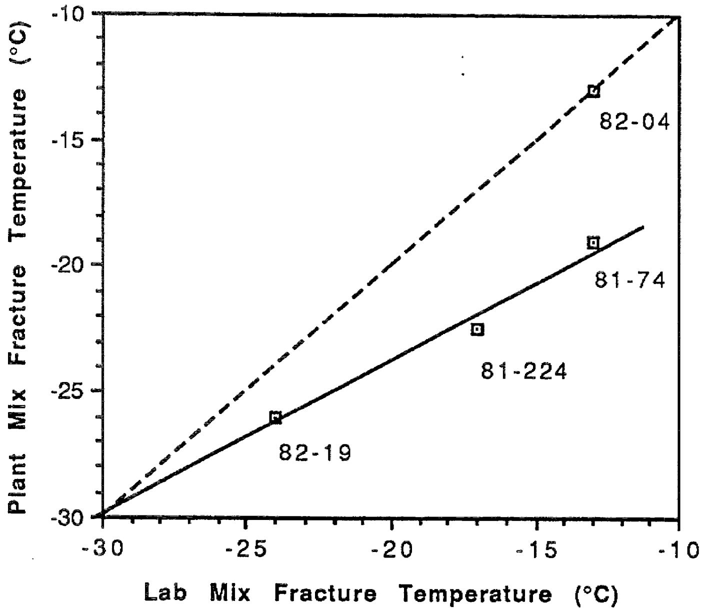
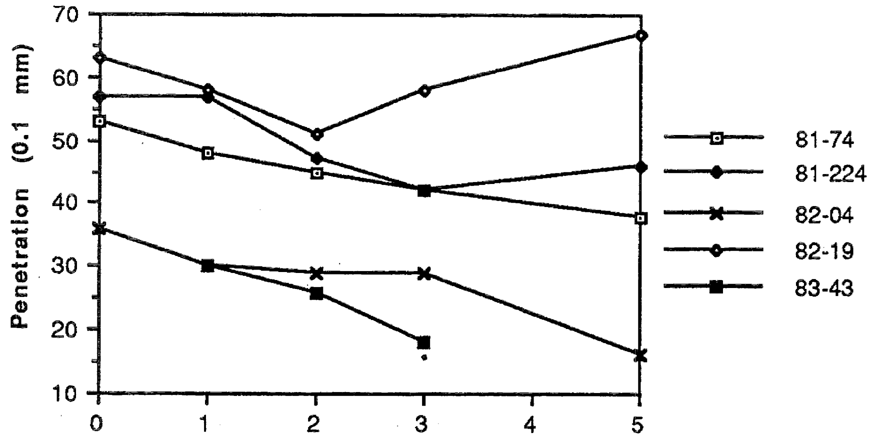


Fig. 6: Effect of Pavement Aging on Binder Penetration and Viscosity

(a)



(b)

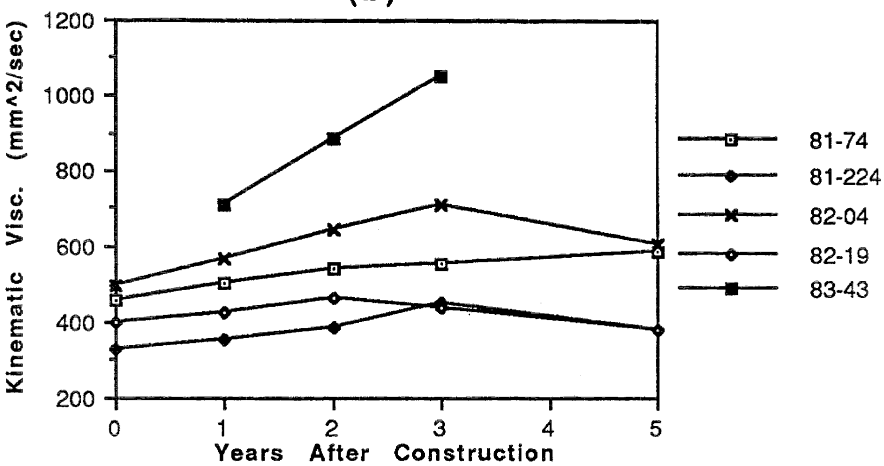
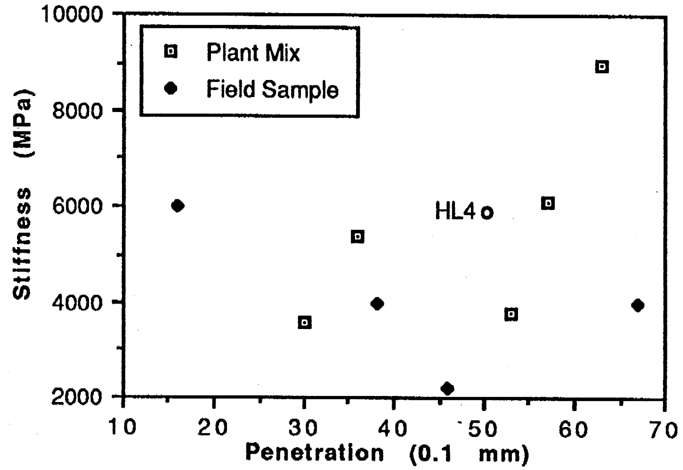


Fig. 7: Plant Mix and Field Sample Stiffness at Fracture Temperature Versus Recovered Penetration and Kinematic Viscosity

(a)



(b)

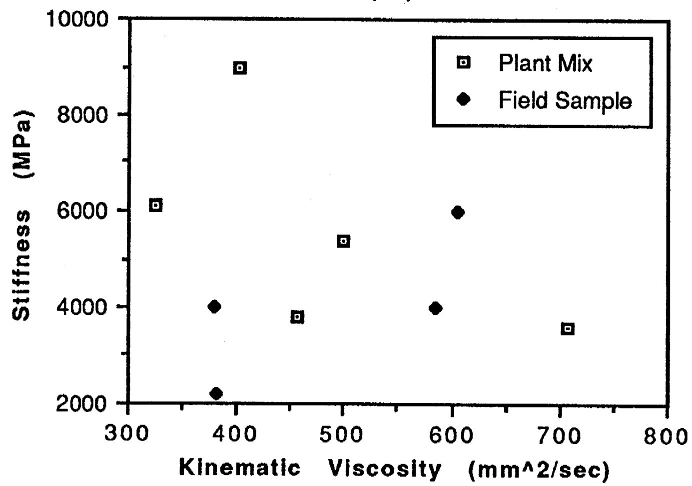
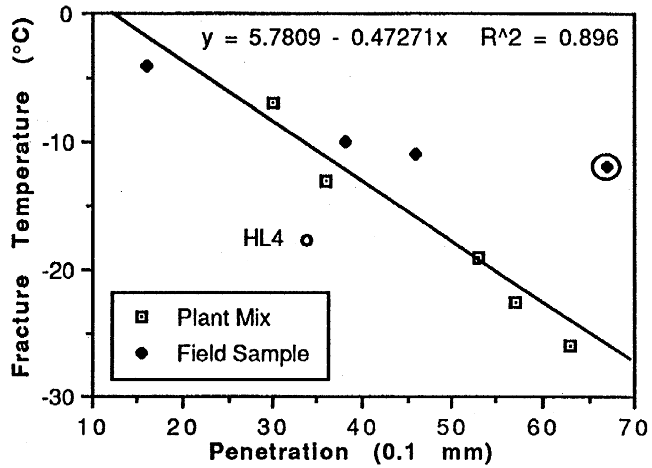


Fig. 8: Plant Mix and Field Sample Fracture Temperature Versus Recovered Penetration and Kinematic Viscosity

(a)



(b)

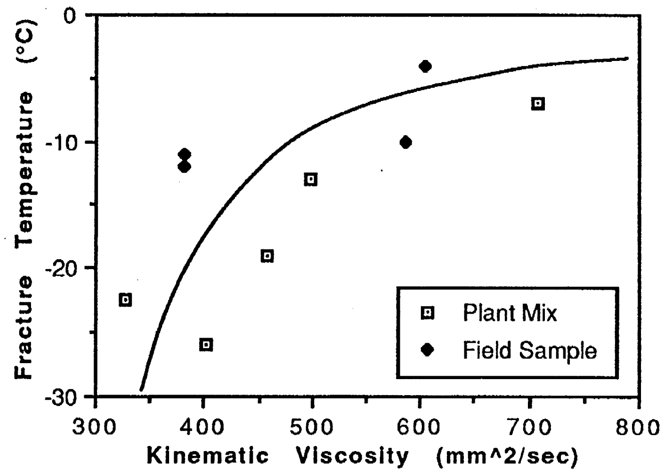
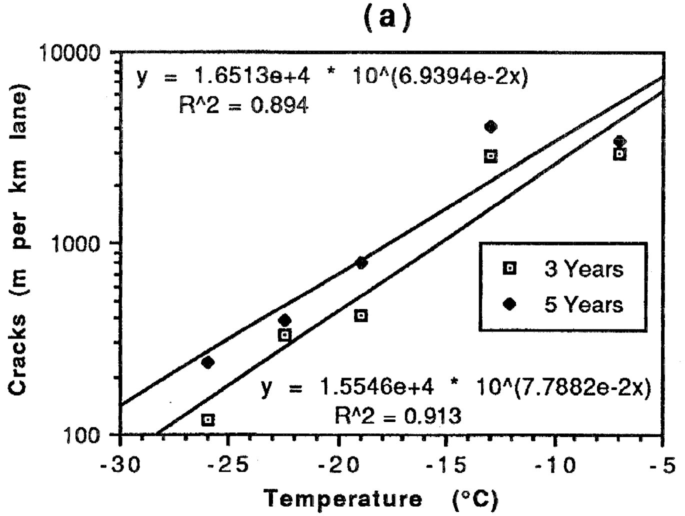


Fig. 9: **Total Length of Cracks Versus Fracture Temperature (FT) and Stiffness at FT of Hot Mix at Construction**



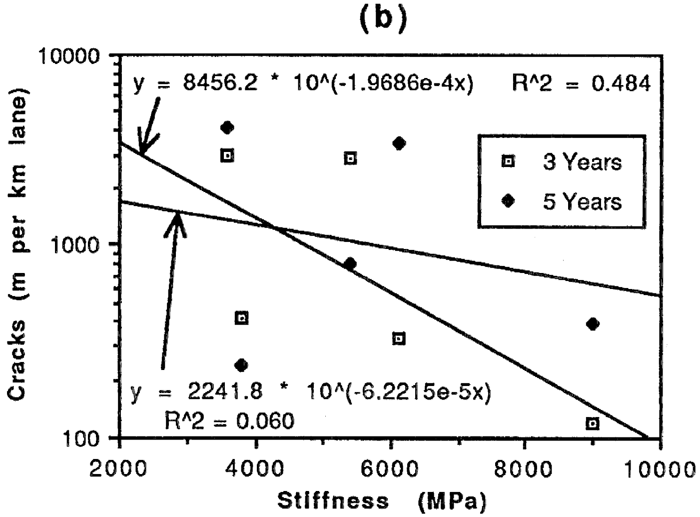
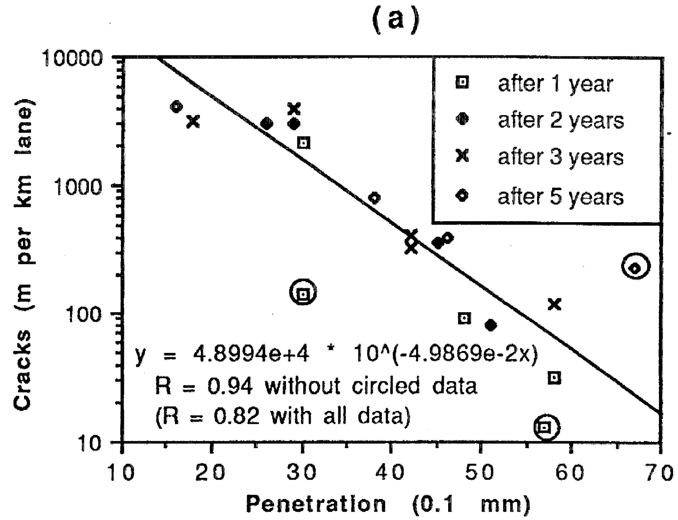


Fig. 10: **Total Length of Cracks Versus Recovered RHM Penetration and Kinematic Viscosity**



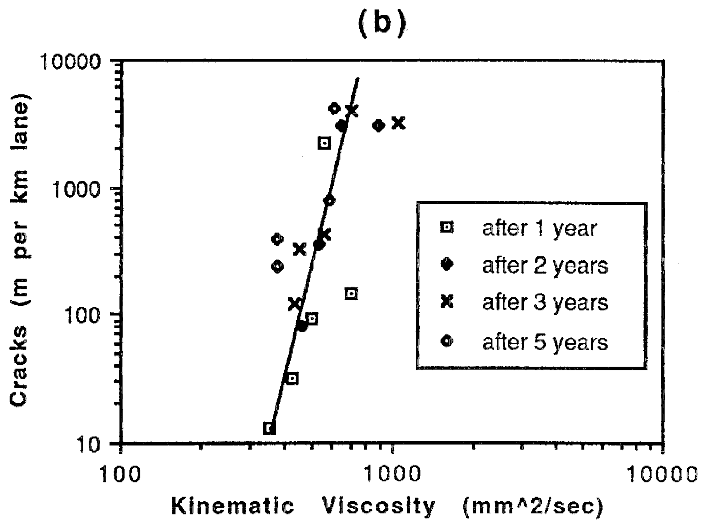


Fig. 11: Total Length of Pavement Cracks versus Recovered RHM Penetration at Construction

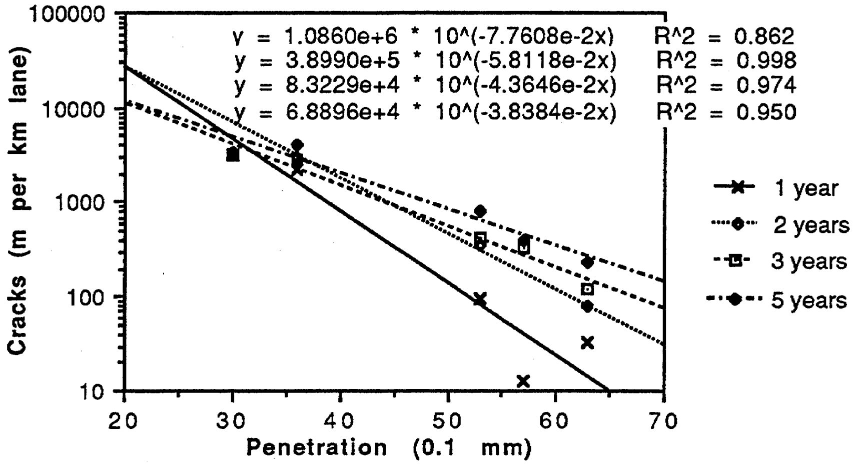


Fig. 12: Stiffness Modulus at Various Fracture Temperatures