Study of Transverse Cracking in Flexible Highway Pavements in Oklahoma

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The results of a research study to determine the nature and extent of transverse cracking on Oklahoma flexible pavements and to investigate the possible causes of this form of distress are reported and discussed. Nine test sites with various degrees of cracking were selected for comprehensive study. An indirect tensile-splitting test apparatus was developed to evaluate the low-temperature tensile properties of asphalt concrete cores obtained from the test sites. Use was made of the concept of the stiffness modulus to characterize the low-temperature behavior of recovered asphalt cements and mixtures. Measured tensile properties of field core samples, particularly tensile strains at failure, were satisfactorily correlated with the observed degree of cracking. The occurrence of transverse cracking was found to increase as failure strain decreased and failure stiffness increased. In addition, the stiffness moduli of recovered asphalts were significantly correlated with the severity of cracking in pavements at the test sites. As the stiffness of the asphalt cements increased, the degree or amount of cracking increased.

Transverse cracking is a serious highway-performance problem in Oklahoma. Open cracks permit the ingress of surface water, which can cause stripping in the asphalt-bound materials and softening of the subgrade. In extreme cases, depression occurs at these transverse cracks because of subgrade softening or pumping of fine materials or both, and secondary cracks develop parallel to the main crack.

Pavement surfaces with this kind of cracking must be repaired to prevent further deterioration and maintain the safe, smooth riding quality desired by the motoring public. If these cracks are not properly sealed, the surface condition of the pavement may deteriorate to the point that complete resurfacing is required long before the design life of the pavement is reached. Frequently, this expensive solution is unsatisfactory because these cracks tend to reflect through the new surfacing in a short time if they have not been adequately sealed before overlaying.

POSSIBLE CAUSES OF AND MAJOR FACTORS IN TRANSVERSE CRACKING

Transverse cracking can result from a variety of causes. However, many of the studies conducted in different parts of the United States and Canada (1, 2, 3, 4, 5) concluded that this type of cracking could be attributed to the thermal stresses that develop in the pavement as a result of temperature changes, particularly changes in the low-temperature range. One of the most significant variables was found to be the consistency-related characteristics of the bitumen used in the surface layer (1). In addition, factors of pavement age, subgrade type, thickness of the asphalt layer, and traffic loads have been directly related to this problem. It is important to realize that these different factors do not necessarily act independently but may be combined. For instance, the combination of traffic loads with a drop in temperature in an asphalt concrete pavement could create stress of sufficient magnitude to cause a transverse fracture of the pavement.

METHOD AND SCOPE OF STUDY

The primary objective of this study was to determine the nature and extent of transverse cracks on selected Oklahoma pavements and investigate the causes of this form of distress. The study dealt primarily with the bituminous components of the pavement and their influence on or contribution to transverse cracking.

Nine test sites with various degrees of cracking were selected for comprehensive study. Mapping and counting techniques were used to determine the severity of cracking at each site. Pavement cores 152.4 mm (6.0 in) in diameter that spanned newly developed cracks were obtained to study the depth of penetration of these cracks in the asphalt pavement layers. Other cores 101.6 mm (4.0 in) in diameter were taken at random locations in the vicinity of the transverse cracks for further laboratory testing. The low-temperature tensile properties of these 101.6-mm core specimens were determined by using an indirect tensile-splitting apparatus. The asphalt binder was recovered from the respective core specimens, and tests were performed to evaluate the rheological properties of the recovered asphalts. Use was made of the concept of the stiffness modulus to characterize the behavior of the recovered asphalt cements and field core specimens at low temperatures. Test results were analyzed, and correlations with the observed degree of pavement cracking were made by using the statistical analysis system (SAS) computer program (6).

EXPERIMENTAL PROGRAM AND TEST PROCEDURES

The experimental program of this study was divided into two phases: (a) surveying and core sampling at the field test sites and (b) laboratory testing to evaluate the properties of the respective field core samples.

Field Testing Program

Pavement test sites where various degrees of cracking had occurred were selected so that the contributing factors at the sites could be identified and compared. A total of nine test sites were selected for detailed investigation. These selected sites included four sections on US-177 and five sections on I-35 and I-40 in Oklahoma. Two of the Interstate sites had almost no cracking and were chosen for comparative purposes. Information on pavement, base, and subgrade soil at the respective test sites is given in Table 1.

At each test site, a 152-m (500-ft) length of pavement that satisfied established requirements for safe sight distance was chosen for detailed surveying, counting, and coring of cracks. Transverse crack patterns were sketched on an appropriate field data sheet. These cracks were classified according to type (multiple, full, half, and part), and the number of various types of cracks per 152 m of pavement length was used to establish a cracking index (CI) for each test site (7). Some of the "part" cracks were closely inspected, and suitable ones were chosen for further study. Large-diameter cores that

Table 1. Test site data.

Test Site	Highway	Original Surfacing Depth (mm)	Original Surfacing Type[a]	Overlay Depth (mm)	Overlay Type	Overlay Date	Base Depth (mm)	Base Type	Subgrade AASHO Classification	Subgrade Plasticity	CI
1	US-177	38.1 76.2	C A	38.1	C	1972	203.2	Sand asphalt	A-6(10)	LL = 34, PI = 14	6.5
2	US-177	38.1 76.2	C A	38.1	C	1972	203.2	Stabilized aggregate	A-4(5)	LL = 24, PI = 6	10.5
3	US-177	38.1 76.2	C A	38.1	C	1971	203.2	Stabilized aggregate	A-7-6	LL = 42, PI = 21	15.5
4	US-177	38.1 76.2	C A	38.1	C	1971	203.2	Stabilized aggregate	A-2-4	NP	24.5
5	I-35	38.1 76.2	C A				304.8	Stabilized aggregate	A-6(2)	LL = 27, PI = 12	2.0
6	I-35	38.1 76.2	C A	38.1	C	1971	304.8	Sand asphalt	A-4(5)	LL = 25, PI = 8	9.0
7	I-35	38.1 76.2	C A	38.1	C	1972	254.0	Black base	A-6(10)	LL = 36, PI = 16	20.0
8	I-35	38.6 76.2	C A				304.8	Stabilized aggregate	A-6(2)	LL = 24, PI = 11	0.0
9	I-40	38.6 76.2	C A	38.1	C	1972	254.0	Black base	A-2-4	NP	0.5

Notes: 1 mm = 0.0394 in.
LL = liquid limit, PI = plasticity index, and NP = nonplastic.
[a]Oklahoma Department of Transportation asphalt concrete mixtures.

spanned these cracks were obtained in an attempt to determine the mechanism of transverse cracking.

Another part of the field study consisted of securing pavement core samples for laboratory testing. In order to consider the effect of traffic densification on the asphalt paving materials, cores were taken from the pavements at both wheel-path and non-wheel-path locations. The tensile properties of these core samples were to be evaluated at three different low temperatures and, to increase the reliability of the results, three test replicates for each combination of location and temperature were needed. Thus, eighteen 101.6-mm (4.0-in) diameter core specimens were taken from each test site. Randomization principles were used to select the locations of these field cores within the chosen 152-m (500-ft) length of pavement.

Laboratory Testing Program

Careful examination of the large-diameter cores taken at recently developed cracks revealed that the majority of these "beginning" cracks did not extend through the pavement matrix. Apparently, the cracks had originated at the surface and had propagated to only a limited depth in the underlying layers.

Based on the hypothesis that these transverse cracks were caused by tensile forces developed in the pavement surface as a result of the low-temperature response of the asphalt paving mix, the research approach was directed toward evaluating the low-temperature behavior of the asphalt materials and mixtures and correlating this behavior with actual field cracking data, i.e., the cracking indexes of the pavement sections. The tensile properties of the field samples were determined over a range of low temperatures: 0°C, -5°C, and -10°C (32°F, 23°F, and 14°F). Some preliminary tensile-splitting tests of field specimens were done at a lower temperature [-20°C (-4°F)]. However, at this temperature a very brittle behavior was observed, and splitting occurred with little or no deformation of the specimens.

Indirect Tensile-Splitting Test

The tensile-splitting test apparatus used in the study is described in detail elsewhere (8). A load cell was used to measure applied compressive loads, which were applied at a rate of 1.52 mm/min (0.06 in/min). Horizontal deformation of the specimen was measured by series-connected linear variable differential transducers (LVDTs). The output signals from the load cell and differential transducers were fed to an X-Y recorder that plotted a continous load-deformation trace. From this load-deformation trace, the total horizontal deformation (X_{TF}) and the corresponding maximum load causing failure (P_{max}) were determined. Tensile strength was calculated as follows:

$$\sigma_{TF} = 2P_{max}/\pi td \qquad (1)$$

where

σ_{TF} = tensile strength (MPa),
P_{max} = maximum load causing failure (N),
t = thickness of the specimen (mm), and
d = diameter of the specimen (mm).

Total tensile strain at failure for a 101.6-mm (4.0-in) diameter specimen with a 12.7-mm (0.5-in) curved loading strip was determined as follows (9):

$$\epsilon_{TF} = X_{TF}\,[(0.1185\nu + 0.038\,96)/(0.2494\nu + 0.067\,30)] \qquad (2)$$

where

ϵ_{TF} = total tensile strain at failure (mm/mm),
X_{TF} = total horizontal deformation at failure (mm), and
ν = Poisson's ratio of the asphalt concrete material.

Previous studies have shown that Poisson's ratio for asphalt concrete materials varies between 0.25 and 0.35 and averages 0.3 (9). By substituting this value in Equation 2, the total tensile strain at failure can be expressed as

$$\epsilon_{TF} = 0.524\ X_{TF} \qquad (3)$$

From these relations, ultimate failure stiffness (S_{TF}) was computed as the ratio between tensile strength and total tensile strain at failure. It was felt that failure stiffness might be a better indicator of the low-temperature behavior of the material since it combines both tensile strength and failure strain responses.

Evaluation of Field Core Samples

The field specimens used in the tensile-splitting tests were obtained by cutting the surface layer, which was approximately 50.8 mm (2.0 in) thick, from the cores with a concrete saw. Before testing, the dimensions of these specimens were measured and the bulk specific gravity was determined. After testing, the theoretical maximum specific gravity of the surface mixtures in the field specimens was determined by using ASTM D 2041.

The asphalt binder from these specimens was extracted by using 1,1,1-trichloroethane in accordance with method B of ASTM D 2172, and the asphalt cement was recovered from the extraction solution by using a modification of ASTM D 1856. Initial studies indicated that the recovered asphalt was excessively hardened during the standard recovery procedure. Tests made on asphalt samples of known penetration grades showed that this hardening could be minimized by reducing the final test temperature to 155° C (311°F) and maintaining it for only 6 min.

The recovered asphalts were tested for penetration, ring-and-ball softening point, and kinematic and absolute viscosity in accordance with standard American Society for Testing and Materials (ASTM) test procedures. These properties were used in calculating the stiffness moduli of the respective asphalts and asphalt-aggregate mixtures according to McLeod's approach (10).

DISCUSSION OF RESULTS

Tensile Properties at Low Temperatures

The results of tensile-splitting tests were statistically analyzed to test for evidence of real differences in the observed values and to estimate the magnitude of such differences. A significance level of 0.05 was considered the criterion for the rejection or acceptance of the hypothesis of no differences. Correlation studies were performed to investigate the general trend of the relation between each of the tensile properties and the cracking index. To minimize the effect of variation in material properties, these correlation studies were made separately on the test results for the wheel-path and non-wheel-path specimens. A Hewlett-Packard calculator plotter (model 9862A) was used to plot the regression lines, and the coefficients of correlation and determination (r and R^2) for the first- and second-degree polynomials respectively were determined by using the SAS computer program.

The tensile-splitting test results (8) were used as input data for a statistical analysis of variance. The results of this analysis of variance and of the correlation studies are discussed for the three tensile properties investigated.

Tensile Strength

Analysis of variance of all test results indicated a strong evidence of locational (wheel-path and non-wheel-path) differences in values of tensile strength. The observed significance level $\hat{\alpha}$ was 0.0014. The tensile strength of wheel-path specimens was considerably greater than that of non-wheel-path specimens. This can be attributed to the effect of the relatively higher pavement densities developed under traffic loads.

Test temperature had a very significant effect on the tensile strength value ($\hat{\alpha}$ = 0.0001). Average tensile strengths at -10° C (14° F) were noticeably higher than those at -5° C and 0° C (23° F and 32° F) respectively. These results indicated the general behavior of the asphalt concrete mixtures at low temperatures; i.e., as temperature decreased, the mixture became rigid, lost some of its plasticity, and behaved in an elastic manner. Consequently, an increase in average tensile stress at failure could be expected.

The analysis of variance also showed that the greater variation in the tensile strength values was attributable to differences in the properties of the test sites ($\hat{\alpha}$ = 0.0001). In general, higher tensile strengths were observed for the Interstate sites (sites 5 through 9). This may indicate the importance of the quality and adequacy of pavement design and construction procedures to service behavior at low temperatures.

Figures 1 and 2 show the regression lines, correlation coefficients (r), and corresponding observed significance levels ($\hat{\alpha}$) of the relation between tensile strength and cracking index for wheel-path and non-wheel-path specimens. In general, test sites with a high degree of cracking showed lower tensile strengths. However, a considerable scattering of tensile strength values was observed, and this reduced the observed correlation coefficients. Much of this data scatter was considered to be a result of the natural nonhomogeneity of the paving materials. It has also been suggested that the scatter might have been caused by the fact that the test temperatures were considerably higher than the low temperatures that initiated the transverse cracking in the pavement.

Tensile Strain at Failure

The calculated tensile strains at failure for wheel-path specimens were considerably higher than those for non-wheel-path specimens. The analysis of variance of the test results showed that the observed significance level associated with differences in location was 0.0042. It generally appeared that the average tensile strain at failure significantly decreased as temperature decreased. The observed significance level was 0.034. This substantiated the effect of service temperature on the behavior of the asphalt concrete mixtures; i.e., failure strains at low temperatures are related to the elastic response of the asphalt mixture.

Again, tensile strains at failure differed considerably between test sites ($\hat{\alpha}$ = 0.001), and greater tensile strains at failure were associated with the higher quality pavements at the Interstate test sites.

Results of the correlation analysis indicated a strong relation between tensile strains at failure and observed degree of cracking (Figures 3 and 4). The coefficients of determination (R^2) associated with this relation were considerably higher than those for the tensile strength values. The occurrence of transverse cracking was found to increase as the tensile failure strain of the pavement specimens decreased. This suggests that resistance to cracking at any low temperature may be a function of the strain

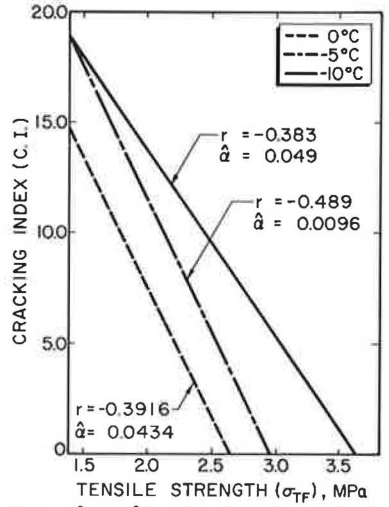

Figure 1. Tensile strength of wheel-path specimens versus cracking index.

Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

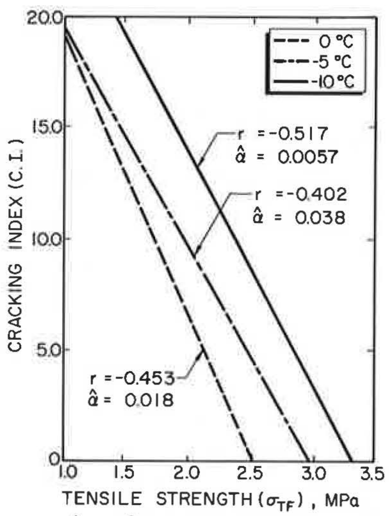

Figure 2. Tensile strength of non-wheel-path specimens versus cracking index.

Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

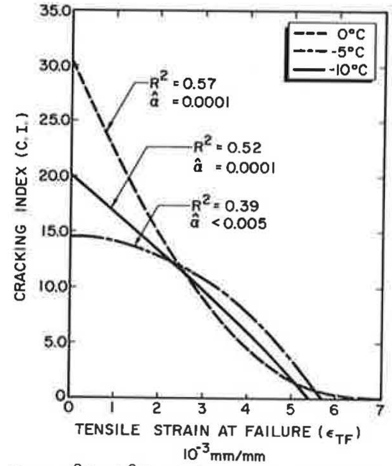

Figure 3. Tensile failure strain of wheel-path specimens versus cracking index.

Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 mm = 0.039 in.

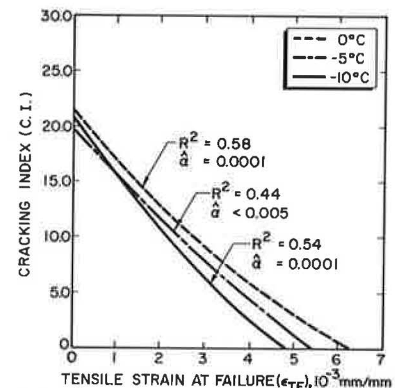

Figure 4. Tensile failure strain of non-wheel-path specimens versus cracking index.

Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 mm = 0.039 in.

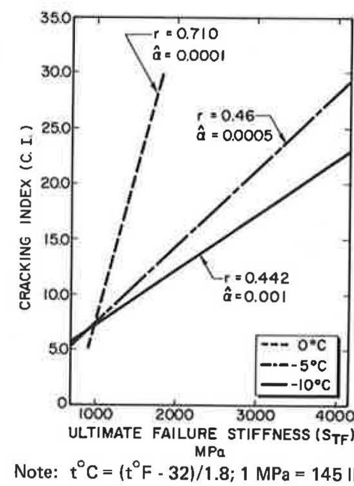

Figure 5. Ultimate failure stiffness versus cracking index.

Note: $t^\circ C = (t^\circ F - 32)/1.8$; 1 MPa = 145 lbf/in^2.

capability of the asphalt concrete mixture at that temperature. It also appears that a permissible, or standard, failure strain for pavement mixtures used in a given geographic region could be established.

Ultimate Failure Stiffness

The analysis of variance did not indicate that a difference in location influenced the failure stiffness (S_{TF}) values. However, a strong evidence of temperature differences was indicated ($\hat{\alpha} = 0.005$). In general, higher average values of failure stiffness were observed at lower temperatures. These results emphasized the earlier findings and indicated that the test sites were significantly different with respect to their ultimate failure stiffness responses.

Results of the correlation analysis, shown in Figure 5, indicated that the cracking index was proportional to the ultimate failure stiffness at all test temperatures and that test sites with a high degree of cracking generally exhibited the higher failure stiffness values. However, it should be noted that the determined correlation coefficients were relatively small. The best correlation between failure stiffness and degree of cracking was that at 0° C (32° F).

Stiffness Moduli of Recovered Asphalt Cements and Mixtures

McLeod's method (10) was used to calculate the stiffness moduli of recovered asphalt cements and mixtures. A study of climatological data over a 73-year period (11) revealed that the lowest minimum air temperature recorded in Oklahoma City was -27.22° C (-17° F). Oklahoma City is more or less centrally located in the state as were the pavement sites studied. Based on temperature data reported in another research study (12), the temperature

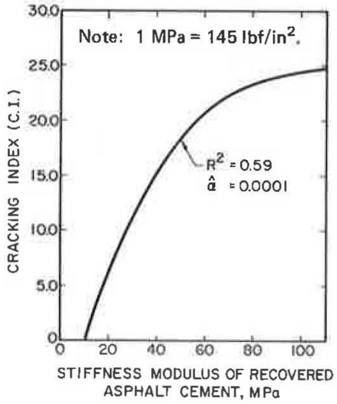

Figure 6. Stiffness moduli of recovered asphalt cements versus cracking index.

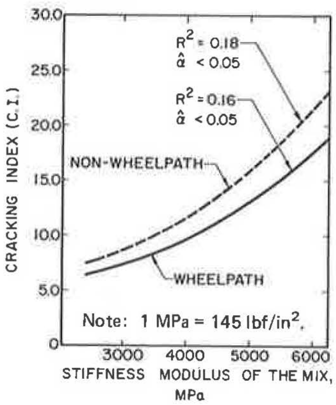

Figure 7. Stiffness moduli of field mixtures versus cracking index.

at a pavement depth of 50.9 mm (2.0 in) was about 3.9° to 4.4°C (7° to 8°F) higher than the air temperature. Consequently, modulus of stiffness values were calculated at a temperature of -23.33°C (10°F).

Correlation studies indicated that the low-temperature stiffness moduli of the recovered asphalt cements could be related to the observed degree of cracking or the cracking index (Figure 6). Pavement sections with a high degree of cracking were generally those that had the stiffer asphalt cements in the surface layer. Only site 7 on I-40 was an exception to this trend. The cracking index of this section was 20.0, but the stiffness modulus was relatively low. It is possible that the high frequency of cracking at this site was associated with a subgrade problem or other related factors. If the data for this site are disregarded, the coefficient of determination goes up to 0.82. This is a remarkably high value considering the variables involved in this generalized relation.

A similar relation was found for the stiffness moduli of the field mixtures (Figure 7). However, the coefficients of determination associated with this relation were smaller than those of the previous one. This can be attributed to the variation in the other mix properties of the surfacing at the individual test sites—i.e., variation in asphalt content, specific gravity, and percentage of density. Again, if the stiffness moduli of the mixtures at site 7 are disregarded, the coefficients of determination increase to 0.31 and 0.32 respectively for the wheel-path and non-wheel-path relations.

The previous findings were substantiated by the results of another research study performed in Texas (13). Transverse cracking in pavements in central and west Texas was significantly related to hardening of the asphalt cement. As asphalt cement hardens with time, a considerable increase in its stiffness occurs. This results in an increased low-temperature cracking susceptibility of the pavement surface.

CONCLUSIONS

The following major conclusions were derived from this study:

1. Examination of recently developed transverse cracks revealed that, in most cases, the cracks had originated at the pavement surface. Thus, the major cause of these cracks appears to be the cold-temperature contraction of the asphalt concrete surface layer.
2. Temperature had a highly significant effect on the measured tensile properties of the paving mixtures. As temperature decreased, tensile strengths and failure stiffness remarkably increased and tensile strains at failure decreased. This is primarily a result of the increase in stiffness of the asphalt binder.
3. A satisfactory correlation was found between the results of the tensile-splitting tests and the observed degree of cracking. The occurrence of transverse cracking was found to increase as failure strain decreased and failure stiffness increased.
4. A permissible or standard failure strain can be established for a pavement mixture in a given geographic region. Such a value could be used in future mix design procedures in which asphalt viscosity, asphalt content, and aggregate gradation are modified to meet design criteria for failure strain.
5. The stiffness moduli of recovered asphalts, determined at the expected minimum temperature in central Oklahoma, were significantly correlated with the cracking indexes of the pavement test sites. The stiffer or harder the asphalt cement in a pavement was, the greater was the degree of transverse cracking.

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Minnesota Heat-Transfer Method for Recycling Bituminous Pavement

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A method for hot-mix-recycling of in-place bituminous pavement that was developed for use on a roadway project in Maplewood, Minnesota, is described. An urban four-lane street was reconstructed to a four-lane divided roadway with turn lanes and transit bus turnouts. The bituminous material was salvaged and recycled along with the upper 0.33 m (1 ft) of gravel base. The old material was scarified, picked up, and hauled to the mixing plant where it was run through a normal three-crusher plant (one jaw and two rolls) and stockpiled. The upper 0.33 m of gravel base was loaded off the roadway, hauled to the plant site, and stockpiled. The salvaged gravel was conveyed from the stockpile to the dryer where it was heated to 232°C to 260°C (450°F to 500°F). The unheated, crushed, salvaged bituminous material was conveyed directly from the stockpile to the weigh hopper above the pug-mill mixer. For the base course, 50 percent salvaged bituminous material was blended with 50 percent salvaged gravel at the mixer for 20 s; then 3 percent asphalt was added and wet-mixed for 30 s. The proportions for the binder course were 40 percent salvaged bituminous, 60 percent salvaged gravel, and 3.5 percent asphalt. The paving was done by use of conventional equipment and conventional methods. Test results for the finished product and the performance of the pavement to date (winter, spring, and summer) indicate that the structure is comparable to a conventional full-depth asphalt base.

Recent developments in the road-building industry have prompted a reassessment of the recycling of bituminous pavements and base aggregates on construction projects. These developments were the rising cost of asphalt, an awareness of the need to conserve finite deposits of nonrenewable natural resources, and a difficulty in finding environmentally and politically acceptable disposal sites for the debris that results from construction and demolition.

Recycling of in-place road materials is not a new concept. In the past, however, when pavement surface and base materials were reused in selected subgrade or base applications, no benefit was realized from the old asphalt binder. Whether the reused material was a gravel base or a pavement surface material, the assigned structural value was no more than that of gravel. A more cost-effective alternative would be to rejuvenate the old asphaltic binder of the in-place bituminous material. Recycling and placing this material as a hot mix rather than an aggregate base would produce a higher strength pavement structure.

The opportunity to hot-mix-recycle came when Raymond Hite, then superintendent of public works in Maplewood, Minnesota, came to the Minnesota Department of Transportation (DOT) for assistance with a project he felt should include the recycling of the in-place bituminous pavement. This project was an urban street of conventional flexible pavement design that was to be reconstructed as a full-depth asphalt pavement. Shortly thereafter, the Minnesota Local Road Research Board authorized funds for setting up and evaluating an urban and a rural bituminous recycling project.

In reviewing the experience of other agencies, which used direct heat in the softening and mixing process, it was noted that the major problem was air pollution in the form of smoke—a problem caused, for the most part, by the burning of asphaltic cement. In at least one other case, a patented process had solved the air pollution (smoke) problem. That process called for a new, completely redesigned drum mixer that used an indirect method of heating the salvaged bituminous mix. A proprietary softening agent was also added to the salvaged bituminous material during mixing operations. The use of that process would have meant buying or leasing the indirect-heating drum mixer and the payment of royalties. It was determined that this would not be cost-effective for the Maplewood project, which called for only 18 144 Mg (20 000 tons) of recycled mix. Coupled with these problems of pollution and expense was the fact that the recycling done thus far was limited to quite low production rates at the mixing plants.

Thus, it was concluded that on this project, because of