Road Test to Determine Implications of Preventing Thermal Reflection Cracking in Asphalt Overlays

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In predominantly cold climatic regions, thermal cracking of asphalt pavements and its reflection through bituminous resurfacings is a problem of great concern to the pavement engineers. Reflection cracking causes poor riding quality prematurely, reduces the useful life of a resurfacing, requires accelerated maintenance, and results in an uneconomic use of physical and fiscal resources. Over the years, many treatments have been tried to minimize the reflection cracking in bituminous resurfacing. These treatments have exhibited varying degrees of success; however, none has been consistently successful under all conditions. In Ontario, Canada, eight test sections were constructed in 1971 to determine a viable alternative to the predominantly used conventional resurfacing. A special feature of this experimental road is the two test sections in which the existing asphalt surface was pulverized and used with or without additional asphalt binder as a base for the resurfacing. In this paper, the phenomenom of thermal cracking and its mechanisms and manifestations are discussed. The experimental road is described and the performance of its various test sections over the past 5 years is documented. An economic analysis is conducted in which the trade-offs between the initial construction and the future maintenance costs of various treatments are compared to the costs of a conventional resurfacing. This analysis concludes that pulverization of the existing pavement surface and use of that surface as a base for resurfacing is the most viable alternative to a conventional resurfacing. The paper also describes three full-scale contracts, totaling about 50 km (31 miles), in which treatment was recently used in Ontario.

In most Canadian provinces and in the northern United States, non-load-associated transverse cracking caused by severe climatic conditions is a predominant form of distress on bituminous pavements. This form of cracking occurs mainly in the bituminous-surface layers when tensile stresses caused by rapid drops in temperatures during the winter exceed the tensile strength of the bituminous material.

Effective rehabilitation of hundreds of kilometers of these cracked pavements is a task that challenges pavement engineers. Bituminous resurfacing has been predominantly used in the past to rehabilitate these pavements; however, this kind of resurfacing has not been a satisfactory form of rehabilitation because the cracks existing in the original pavement reflect through the resurfacing. This cracking is called reflection cracking and is caused when the cracked underlying pavement contracts over subsequent winters and restraint stresses along the underside of the resurfacing are set up. These restraint stresses create high tensile stresses immediately above the existing cracks and therefore lead to a fracture of the resurfacing generally above the crack.

Thermally induced cracks begin as hairline cracks in the first winter and slowly widen with time. As shown in Figure 1, these cracks are partial transverse cracks at first but slowly, during the subsequent winters, extend over the full width of the pavement. These cracks are generally at right angles to the pavement centerline, but sometimes take a different form when two partial cracks are joined by a small longitudinal crack, or when a main crack manifests into multiple or alligator-type cracking.

Figure 2 shows two extreme cases of thermal cracking in Ontario. As shown by the solid lines in this figure, the thermal cracks in new pavements are widely spaced for the first 3 or 4 years, after which one of the following two conditions occurs.

1. Cracks per kilometer progressively increase every winter in proportion to the severity of the winter. In extreme cases, these cracks reach a spacing of 1.5 m (5 ft) or less in 12 to 15 years.
2. Cracks per kilometer increase sharply in the fourth or fifth winter to a spacing of approximately 30 m (100 ft), after which the spacing remains practically constant.

The thermal cracks are hairline for the first 4 to 5 years, after which they progressively widen and eventually become 10 to 20 mm (½ to ¾ in) wide. The cracks are partial or full-width singular for the first 4 or 5 years, after which secondary cracks may form, and, during the twelfth to fifteenth year, severe spalling and alligatoring may be observed.

As shown by the dotted lines in Figure 1, the two crack patterns described above progress at a much faster rate in the case of a resurfaced pavement. In the first case, complete reflection of the cracks in the underlying pavement may occur in 5 to 6 years, with most of the reflection occurring in the initial winters. As in new pavements, cracking is proportional to the severity of winter: The severer the winter, the greater the number of new cracks that develop during that year. In the second case, complete reflection may take place in the first winter after the resurfacing.

In new as well as in resurfaced pavements, the primary distress mode of cracking is generally manifested by many types of secondary distresses. Water and deicing salts infiltrate through the cracks and soften the base material underneath. This infiltration results in partial loss of support that often leads to multiple or even alligator-type cracking around the main crack.

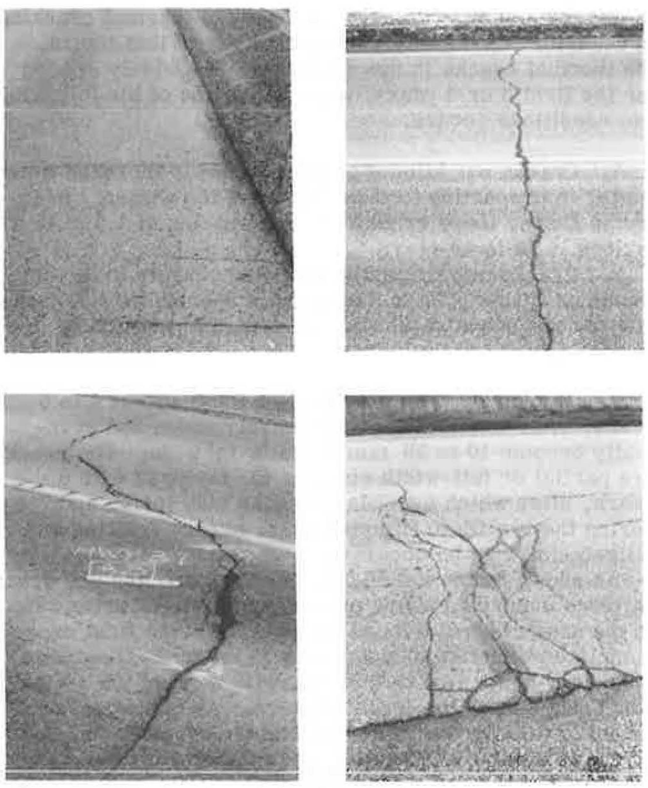
The softened base around the crack, especially during the winter months when deicing solutions cause localized thawing of the base, also results in a depression around the crack called dipping of the crack. At other times, water entering the cracks may freeze and form an ice lens below the crack, thus elevating the crack edges. This elevation is called lipping or tenting of the crack. The lipping and dipping of the crack may occur any time after five to seven winters on new as well as resurfaced pavements.

CONSEQUENCES OF REFLECTION CRACKING

The secondary distress manifestations described above result in the following:

1. Poor riding quality at an earlier date, especially during the winter;
2. Accelerated deterioration of the resurfaced pavement;
3. Increased maintenance demand;
4. Inconvenience to the motoring public;
5. Unsafe driving; and
6. Uneconomical use of physical and fiscal rehabilitation resources.

Figure 1. Typical partial and full transverse cracking.



Therefore, it is important that new techniques be explored to reduce reflection cracking so that existing thermally cracked highway pavements can be economically rehabilitated.

CONVENTIONAL SOLUTIONS

Over the years, many different construction techniques and materials have been tried to minimize or eliminate the reflection cracking of bituminous resurfacings. These treatments have shown varying degrees of success: Some have been successful on specific projects; however, none has completely eliminated reflection cracking or has been consistently successful in minimizing reflection cracking. These treatments generally fall in the following three categories:

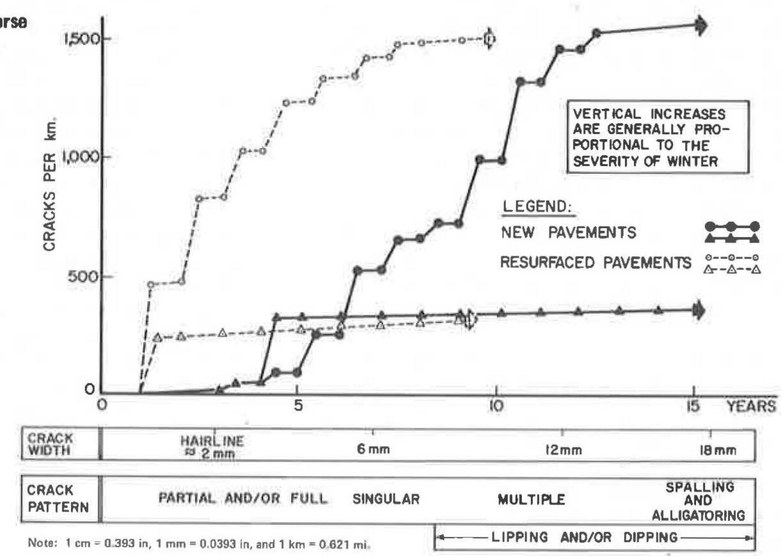
1. Use of improved mixes for resurfacing such as the use of high binder content, softer asphalts, mastic-like mixes such as Gussasphalt, and the use of rubber and polymer-asphalt additives;
2. Use of intervening layers such as granular materials and open-graded asphalt mixes; and
3. Stress-relieving interfaces such as rubber-tire aggregate slurry and thermoplastic rubber that is mixed with asphalt cement and filler.

TROUT CREEK EXPERIMENTAL ROAD

In view of the wide variety of treatments that have been tried but have not been consistently successful for all environmental conditions and the fact that severe winter conditions such as those existing in northern Ontario may have unique influences toward the propagation of reflection cracking, an experimental road consisting of eight treatment sections was constructed in 1971 to study the treatments that may minimize or eliminate reflection cracking in Ontario.

The experimental road site is located about 29 km (18 miles) south of North Bay in Ontario. In the original pavement, the transverse cracks were spaced 1.5 to 3 m (5 to 10 ft) apart, were generally 6 to 12 mm (¼ to ½ in) wide, and were depressed 2.5 to 5.0 cm (1 to 2 in) below

Figure 2. Schematic representation of transverse cracking in new and resurfaced pavements.



Note: 1 cm = 0.393 in, 1 mm = 0.0393 in, and 1 km = 0.621 mi.

the pavement surface. This situation resulted in a rough ride during the winter months. The experimental construction was carried out between August and October 1971.

Cross Sections and Construction Details

Eight treatments were used in the experiment: Four treatments consisted of interlayers between the resurfacing and the old pavement (sections 1 to 4), two treatments consisted of resurfacings over reworked old pavements (sections 6 and 7), one treatment consisted of removing the old pavement surface and replacing it with a new surfacing (section 5), and the last treatment was a control section that consisted of conventional resurfacing over the old pavement (section 8). The cross sections of the various treatments are shown in Figure 3. A standard well-graded mix with a 150 to 200-penetration asphalt was used as a resurfacing for each treatment.

1. Screenings interlayer—Section 1, 152 m (500 ft) long, consisted of a 2.54-cm (1-in) thick layer of crushed-stone screenings, 9.5-mm ($^3/_8$-in) maximum size, that was spread over the old pavement before resurfacing.
2. Granular interlayer—Section 2, 1.07 km (0.67 mile) long, consisted of a 7.6-cm (3-in) thick layer of granular, 22.2-mm ($^7/_8$-in) maximum size, that was spread over the old pavement before resurfacing.
3. Granular interlayer—Section 3, 1.07 km (0.67 mile) long, consisted of a 15.2-cm (6-in) thick layer of granular, the same as in section 2, that was spread over the old pavement before resurfacing.
4. Open-graded binder interlayer—Section 4, 1.96 km (1.22 miles) long, was resurfaced with the first resurfacing (binder) layer that was made of an open-graded mix with 100 percent crushed aggregate.
5. Existing surface replaced—For section 5, 2.9 km (1.8 miles) long, the existing pavement surface was removed and replaced with a new surface.
6. Existing surface pulverized—For section 6, 1.61 km (1.0 mile) long, the old pavement surface was pulverized, relaid on the old granular base, compacted, and used as a base for the resurfacing.
7. Existing surface pulverized and enriched—Section 7, 1.4 km (0.9 mile) long, was similar to section 6 except that the pulverized material was enriched with approximately 3 percent of medium curing-250 (MC-250) cutback.
8. Conventional resurfacing—Section 8, 1.8 km (1.1 miles) long, was a control section because the treatment used in this section has been conventionally used in Ontario for pavement rehabilitation purposes.

The effects of grooves on the performance of the resurfacings were studied by cutting approximately 20 lateral grooves in each of sections 2 through 8. These grooves, 12.7 mm ($^1/_2$ in) thick and 12.7 mm ($^1/_2$ in) deep, were cut across the full width of the finished resurfacing and were filled with hot-poured rubberized joint sealant. At the beginning of each treatment, four grooves were cut at interval spacings of 7.6 m (25 ft), 15.2 m (50 ft), and 22.8 m (75 ft), and seven or eight grooves were cut at random locations where cracks existed in the original pavement.

Observations From Experimental Road Construction

The problems that were encountered during the construction of the various sections are summarized as follows:

1. In section 1, control problems were experienced in maintaining the 2.54 cm (1 in) of crushed stone screenings over the distortions in the existing pavement. One problem was to avoid disturbing this material by the tires of the asphalt paver when the first resurfacing (binder) layer was placed. An increase in the thickness of the binder layer to 5.08 cm (2 in) and a slower paving operation partially reduced this problem.
2. In section 2, control problems were also experienced with the 7.62-cm (3-in) granular layer because this thickness does not uniformly cover the distortions or provide a tight surface that is resistant against traffic. This problem did not exist in section 3, which had a 15.24-cm (6-in) granular thickness.
3. In sections 4, 5, and 8, there were no construction difficulties encountered.
4. In sections 6 and 7, where the existing pavement surface was pulverized and reused, many construction difficulties were encountered but were resolved. In section 6, the old pavement surface was first ripped and windrowed to the side. A small Hammermill pulverizer was initially used to pulverize this material on site. This pulverizer had frequent breakdowns and its teeth wore down considerably each day. This operation was discontinued, and, subsequently, the material was hauled, stockpiled, and pulverized by using a crusher. In section 7, the enriched mix with 4 percent cutback (initially used) would not set up fast enough after placing and was severely distorted when opened to traffic. The material was windrowed to the shoulder and cured with the help of a grader. The amount of MC-250 was reduced to 3 percent so that the residual asphalt content in the mix and the curing time could be reduced.

Performance

Since the construction of the experimental road, each of the eight test sections has been assessed twice a year by the staff of the Ontario Ministry of Transportation and Communications. The results of the crack surveys are shown in Figure 3. As per the last survey in July 1976, section 8 (control), which used the conventional resurfacing, had the maximum cracking [183 cracks/km (294 cracks/mile)]. The cracking in other sections, in terms of percentage of cracking in control section, is as follows:

Section	Percentage of Control Cracking
1 and 4	70 to 80
2 and 3	25 to 35
5, 6, and 7	3 to 10

All surveys conducted to date show that sawn grooves are of no benefit in controlling the reflection cracking. The cracks have appeared within very short distances of these grooves, sometimes even within a few centimeters.

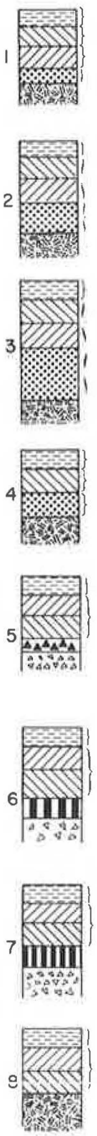
Cost

Based on 1971 bid prices for the experimental project, the construction costs in U.S. dollars of various test sections were computed. Costs varied from a minimum of \$19 453/km (\$31 300/mile) for section 8 (control) to a maximum of \$36 234/km (\$58 300/mile) for section 7, which was 186 percent increase over the control. The costs of other sections, in terms of percentage of control, are given in Table 1.

Conclusions

Based on cracking history, construction difficulties, and construction cost comparison as given in Table 1, it is

Figure 3. Cross sections and cracking history of Trout Creek experimental road.



Section	Cross-section	Type	MAY 72	NOV 72	MAY 73	NOV 73	MAY 74	OCT 74	MAY 75	NOV 75	JUL 76
			*W		W		W		W		W
1	3.17 cm surfacing; 3.17 cm binder; 5.08 cm binder; 2.54 cm screenings; Old Pavement	**T	0	0	0	0	6	6	6	6	131
		**L	0	0	0	0	0	0	0	0	0
2	3.17 cm surfacing; 6.98 cm binder; 7.62 cm granular; Old Pavement	T	0	0	5	6	9	9	9	9	66
		L	0	0	1	1	2	2	2	2	2
3	3.17 cm surfacing; 6.98 cm binder; 15.24 cm granular; Old Pavement	T	2	2	5	5	8	10	10	10	47
		L	0	0	0	0	0	0	0	0	2
4	3.17 cm surfacing; 3.17 cm binder; 3.81 cm open graded mix; Old Pavement	T	45	46	75	78	90	93	99	99	149
		L	0	0	0	0	7	9	21	21	34
5	3.17 cm surfacing; 6.98 cm binder; New Granular; Old Granular; Old Surface Removed	T	1	1	3	3	7	7	8	8	19
		L	0	0	0	0	1	1	2	2	3
6	3.17 cm surfacing; 6.98 cm binder; Old Surface Pulverized; Old Granular	T	0	0	0	0	2	2	2	2	7
		L	1	1	1	1	2	2	2	2	2
7	3.17 cm surfacing; 6.98 cm binder; Old Surface Pulverized and Stabilized; Old Granular	T	0	0	0	0	0	0	0	0	5
		L	0	0	0	0	0	0	0	0	0
8	3.17 cm surfacing; 6.98 cm binder; Old Pavement	T	53	57	85	87	107	107	116	116	183
		L	0	0	0	0	3	3	17	17	42

CRACKS PER km

Note: 1 cm = 0.393 in and 1 km = 0.621 mile.

* W Winter
** T Transverse cracks
** L Longitudinal cracks

Table 1. Summary inferences from Trout Creek test road.

Section	Treatment	1976 Cracking (percentage of control)	Construction Difficulties	Extra Cost[a] (percentage of control)	Further Consideration
1	Screenings interlayer	72	Considerable	27	No
2	7.6-cm granular interlayer	36	Moderate	32	Yes
3	15.2-cm granular interlayer	26	None	67	Yes
4	Open-graded binder interlayer	81	None	0	No
5	Surface replaced	10	None	23	Yes
6	Surface pulverized	4	Moderate	31	Yes
7	Surface pulverized and enriched	3	Moderate	86	Yes
8	Conventional resurfacing	100	None	0	Control

Note: 1 cm = 0.39 in.

[a]Based on 1971 bid prices for the test road sections.

Figure 4. Schematic representation of agency cost components.

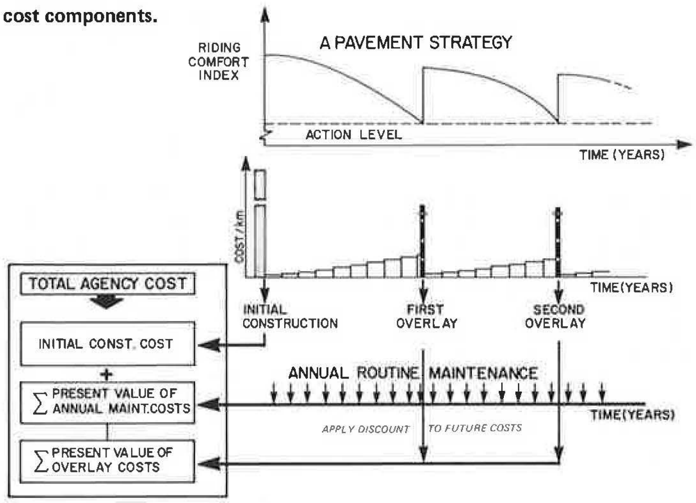


Table 2. Maximum prices for extra life with distortion correction.

Extra Life[a] (years)	Distortion Correction (U.S. $/m²) Severe	Normal	None
AADT >2000			
0	0.60	0.30	0.00
1	0.80	0.50	0.20
2	1.02	0.72	0.42
3	1.21	0.91	0.61
4	1.40	1.10	0.80
5	1.60	1.29	0.99
6	1.75	1.45	1.15
9	2.26	1.96	1.66
12	2.58	2.28	1.99
15	2.93	2.63	2.33
AADT <2000			
0	0.60	0.30	0.00
1	0.78	0.48	0.18
2	0.94	0.65	0.35
3	1.10	0.80	0.50
4	1.26	0.96	0.66
5	1.46	1.16	0.86
6	1.53	1.23	0.93
9	1.88	1.58	1.28
12	2.16	1.87	1.57
15	2.43	2.13	1.83

Notes: 1 m² = 10.8 ft².
This table is based on 1976 U.S. material prices.
[a]In excess of 10 years, which is considered the average life of a conventional resurfacing.

concluded that treatments in sections 1 and 4 are not viable alternatives to conventional resurfacing and should therefore be eliminated from further consideration and analysis.

ECONOMIC ANALYSIS

An economic analysis is conducted to further establish which of the remaining five treatments is the most viable alternative to a conventional resurfacing for field trials through full-scale contracts. Although performance data are being gathered every year at the experimental road site, the interim recommendations already show the benefits from the 5-year experience to date.

Analysis Rationale

The economic consequences of a highway improvement are twofold: the agency costs of initial improvement and subsequent maintenance, and user benefits of operating on such an improved facility. The trade-offs between the cost of initial construction and that of subsequent future maintenance are important to a highway agency. When compared to a conventional resurfacing, if the extra initial cost (Δ_c) of a treatment is completely offset by the future maintenance cost savings (Δ_F), the treatment may be considered a viable alternative. The ratio of Δ_F to Δ_C, as in any benefit/cost analysis, determines the degree of viability for such a treatment.

The user-associated costs and savings, those of motorists' discomfort and delay, also impinge on the determinations of such viability. For this analysis, user delay costs in U.S. dollars have been calculated; however, they have not been included in this paper because they are subject to varying interpretations. A reader may still superimpose the motorists' considerations exogenously on the results of this economic analysis.

As shown in Figure 4, the future maintenance cost in U.S. dollars of a pavement facility has two components: the annual routine maintenance cost and resurfacing or overlay cost. For economic analysis, these costs are calculated as they incur over a certain analysis period, e.g., 30 years, and are then discounted to their current values so that the trade-offs can be analyzed in terms of current dollars.

A prerequisite for calculating future maintenance costs is that the times must be known when such costs are incurred, i.e., the life of initial improvement as well as those of future rehabilitative actions must be predictable. The exact life of these treatments cannot be estimated because the experiment is only 5 years old; therefore, a general analysis was conducted in which the future savings were calculated. It was assumed that a treatment may last any number of years, and this assumption was compared to a conventional resurfacing. The extra initial cost of a treatment can then determine the number of additional years that such a treatment may last so that this extra cost is offset by the future savings. The following example illustrates this concept.

1. Assume an initial life of T_1 years for a conventional resurfacing and t_1, t_2, ... t_i years for a treatment. Calculate for the conventional resurfacing by using initial cost (R_s) and future cost (R_f). Calculate for the treatment by using initial cost (C_s) and future costs (C_1, C_2, ... C_i). The extra initial cost ($\Delta_C = C_s - R_s$) and the future cost savings (Δ_F) respectively are as follows:

$$\Delta_{F1} = R_f - C_1$$
$$\Delta_{F2} = R_f - C_2$$
$$\cdots$$
$$\Delta_{Fi} = R_f - C_i \qquad (1)$$

Plot Δ_{Fi} versus t_i and compare it with Δ_C to determine t_i at which $\Delta_{Fi} = \Delta_C$. The analysis will thus give the additional life [$(t_i - T_1)$] that a treatment should last to justify its extra initial expenditure.

2. Repeat step 1 to study a range in price at which a treatment can be constructed by using different values for C_s, which would also require using different additional lives.

3. Repeat steps 1 and 2 for a range of initial life values (T_1, T_2, ... T_i) for the conventional resurfacing.

Analysis Details

As per the rationale given, Table 2 gives the maximum treatment prices that can be allowed if the corresponding extra lives are obtained. The prices are for two types of facilities: low-volume roads with annual average daily traffic (AADT) less than 2000, and roads with AADT greater than 2000. The major differences in the two cases are the routine maintenance and the resurfacing thicknesses used for future maintenance.

Because pavement distortions have to be corrected to restore a proper crossfall when a conventional resurfacing is applied, the cost of correcting such a distortion has been added to the allowable treatment price for which no distortion correction is generally warranted. Table 2 gives the allowable treatment prices for the following conditions.

1. No distortion correction;
2. Normal distortion correction, i.e., an average of 6.35-mm (¼-in) correction over the entire project; and
3. Severe distortion correction, i.e., an average of 12.7-mm (½-in) correction over the entire project.

The analysis given in Table 2 is for a case in which the same resurfacing thickness will be provided on the treated pavement as well as on the conventionally resurfaced pavement. However, if the resurfacing thickness on the treated pavement can be reduced by a certain amount, the corresponding saving can be added to the allowable treatment price. For this purpose, the cost in U.S. dollars of 1 cm (0.393 in) of hot mix and shoulder upgrading shall be approximately $\$0.47/m^2$ ($\$0.39/yd^2$).

The details of the entire economic analysis leading to Table 2 cannot be described in this paper. However, the unit material prices used in the analysis are given as follows in U.S. dollars: $\$37.93/m^3$ ($\$29.00/yd^3$) for hot mix for resurfacing and $\$9.81/m^3$ ($\$7.50/yd^3$) for granular material for shoulder upgrading.

The use of Table 2 is demonstrated by the following example. Assume that a thermally cracked low-volume facility is to be rehabilitated and the alternatives being considered are

1. A pavement resurfaced with 7.62 cm (3 in) of hot mix and no distortion correction is needed, and
2. A pavement treated (pulverized) and resurfaced with 6.35 cm (2.5 in) of hot mix.

Assume further that the treated pavement may last 3 years longer than the pavement with the conventional resurfacing. By using Table 2, it can be seen that treating the pavement will be a viable solution if pulverization can be achieved at less than $0.50 + (7.62 - 6.35) \times 0.47 = \$1.10/m^2$.

Observations From Economic Analysis

The average 1976 prices in U.S. dollars of various treatments are given in Table 3. Also given are the extra lives required, in excess of conventional resurfacing, that will justify the average treatment prices. By comparing these extra lives with the cracking experience to 1976 at the Trout Creek experimental road, the following general observations were made.

1. The treatment for section 3, 15.2-cm (6-in) granular interlayer, is not a viable alternative because it requires 15 years of extra life (total life is 25 years if a conventional resurfacing lasts 10 years) to be cost-effective.
2. The treatment for section 2, 7.6-cm (3-in) granular interlayer, may be a viable alternative when a pavement is severely distorted; however, in such cases, the construction difficulties of control may prohibit the use of this treatment. Intermediate granular thicknesses such as 10.2 cm (4 in) are also not justified because of the extra life requirement. There is a small difference in cracking performance between treatments 3 and 2.
3. The treatment for section 7, pulverization and enriching, is also not a viable alternative because its extra life requirement is high. A reduction of at least 2.54 cm (1 in) in resurfacing thickness may, however, justify this treatment for a pavement that needs severe distortion correction.
4. The treatment for section 5, pavement surface removal and replacement, is a viable alternative if stockpiling of the old pavement is environmentally acceptable. Another disadvantage in this case is that the total thickness of the pavement structure remains unchanged.
5. The treatment for section 6, pulverization, is a viable alternative to conventional resurfacing. It reuses the existing materials and contrary to the treatment for section 5, the total structure thickness in this case increases. This treatment is the most cost-effective if the resurfacing thickness can also be reduced by 1.27 cm (½ in) or more. It should be noted, however, that such a reduction is only possible if the pulverization operation does not pick up large quantities of the underlying granular material that minimize the effectiveness of the residual asphalt expected from the pulverized pavement surface.

FIELD TESTS

As a result of the above analysis, the Ontario Ministry of Transportation and Communications initiated three full-scale contracts for pulverization totaling about 50 km (31 miles), as shown in Figure 5. These contracts were intended mainly to further investigate various construction difficulties posed by pulverization and explore possible solutions, and to obtain further data on performance of this treatment. The following is a brief description of the three contracts.

Highway 68

Highway 68, near Sudbury, Ontario, is 15.3 km (9.5 miles) long. This pavement, about 15 years old, showed the following pavement conditions prior to pulverization: (a) fair to poor rideability, (b) transverse cracking with 4.5 to 6.0-m (15 to 20-ft) spacing, and (c) moderate lipping. The pavement, 7.6 cm (3 in) thick and 6.7 m (22 ft) wide, was ripped and windrowed to the middle of the driving lane. The broken pavement was hauled to a

Table 3. 1976 treatment prices for extra life with distortion correction.

Section	Treatment	Price Range (U.S. $\$/m^2$)	Average Price (U.S. $\$/m^2$)	Distortion Correction (years)		
				Severe	Normal	None
2	7.6-cm granular interlayer	1.34 to 1.53	1.44	4	6	8
3	15.2-cm granular interlayer	2.68 to 3.06	2.87	14	>15	>15
5	Surface replaced	0.24 to 0.36	0.30	-1	0	1.5
6	Surface pulverized	0.97 to 1.20	1.08	2	4	5.5
7	Surface pulverized and enriched	2.69 to 3.17	2.93	15	>15	>15

Note: 1 cm = 0.39 in and 1 m^2 = 10.8 ft^2.

Figure 5. Three pulverization contracts in Ontario.

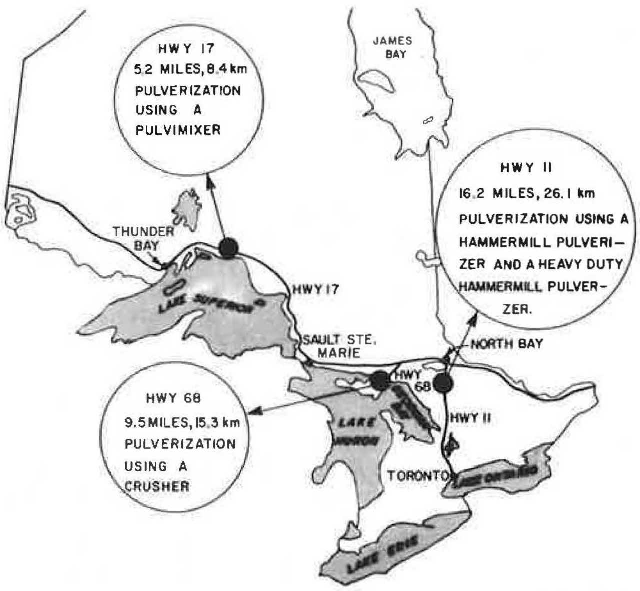


Figure 7. Pulverization by using a Hammermill pulverizer.

Figure 6. Pulverization by using a crusher.

crusher, crushed to a minus 2.54-cm (1-in) size, then hauled back to the road and relaid. The production rate obtained was about 457 m (1500 ft) of two lanes/d. Figure 6 shows the hauling and the crushing operations.

Highway 11

Highway 11, near North Bay, Ontario, is 26.1 km (16.2 miles) long. This pavement, also about 15 years old, showed the following conditions prior to pulverization: (a) fair to poor rideability that became very poor during the spring, (b) transverse cracking with an average 1.5-m (5-ft) spacing, and (c) severe lipping. Approximately 19.3 km (12 miles) of this project were pulverized by using a Hammermill pulverizer. The old pavement with a minimum thickness of 11.4 cm (4.5 in) was ripped one lane at a time and windrowed to the shoulder. This material was then brought inside the lane in small windrows and pulverized. Two or three passes of the pulverizer were generally required to pulverize the material to required size. A production rate of about 213 m (700 ft) of two lanes/d was obtained. Figure 7 shows pavement ripping and pulverization of the pavement.

Approximately 6.4 km (4 miles) of this project were pulverized by using a heavy-duty Hammermill pulverizer. This pulverization was often followed by one pass in the smaller pulverizer, mentioned above, to obtain the required maximum size. Ripping of the pavement was not

Figure 8. Pulverization by using a heavy duty Hammermill pulverizer.

Figure 9. Pulverization by using a pulvimixer.

needed with this equipment. A production rate of about 610 m (2000 ft) of two lanes/d was obtained. Figure 8 shows this equipment in operation.

Highway 17

Highway 17, near Thunder Bay, Ontario, is 8.4 km (5.2 miles) long. This pavement, about 18 years old, showed the following pavement conditions prior to pulverization: (a) fair rideability that became very rough and uncomfortable during the spring, (b) transverse cracking with 0.6 to 3.0-m (2 to 10-ft) random spacing, and (c) severe lipping.

The pavement, 7.3 m (24 ft) wide and 7.6 cm (3 in) thick, was pulverized by a pulvimixer. It was not necessary to rip the pavement because this equipment was used. One to two passes of the pulvimixer were required to pulverize the material to a minus 2.54-cm (1-in) size. A production rate of about 366 m (1200 ft) of 2 lanes/d was obtained. Figure 9 shows this equipment in operation.

CONCLUSIONS

The Trout Creek experimental road has provided valuable information on the performance of various alternatives to conventional resurfacing for rehabilitating thermally cracked asphalt pavements. Of the seven alternatives tried, the economic analysis and other implications indicate that

1. Pulverizing the existing pavement surface and using it as a base for resurfacing is the most viable alternative;
2. Removing and replacing the existing pavement surface is also a viable alternative, if it is environmentally acceptable;
3. Placing interlayers between the old pavement and the resurfacing such as crushed stone screenings, different thicknesses of granular material, and open-graded binder course is not cost-effective; and
4. Enriching the pulverized material and using it as a base for resurfacing is not a cost-effective alternative.

The three full-scale contracts undertaken to date have demonstrated that construction difficulties associated with the pulverization operation can be resolved.

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Analytical Modeling and Field Verification of Thermal Stresses in Overlay

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This paper describes analytical and graphical procedures for computing thermal stresses at joint locations in pavement overlays. Equations and nomographs are used to calculate stresses caused by horizontal and vertical movements of slabs. Both average temperature drop and maximum temperature differential expected in pavement slabs are determined from temperature distribution noted at time of overlay construction. Stresses caused by slab movement are calculated for different overlays. The results confirm that these stresses often exceed the maximum stresses in asphalt-concrete overlays; therefore, reflective cracking occurs when asphalt concrete is laid over jointed pavements.

The movement of pavement slabs under flexible overlays has been well known for its damaging effect on overlays. This effect is usually manifested by the phenomenon of reflective cracking. The slab movements induced by temperature are usually considered from two points of view: One arises from slow changes in average temperature of pavement, and the second arises from quick changes in average temperature of pavement, i.e., a cool night to a hot day and vice versa. In the first case, pavement slabs contract and expand because of a change in the average temperature of pavement. In the second case,