BLOCK 6.0

SELECTION OF THE PREFERRED DESIGN ALTERNATIVE

Example Analysis of Selecting Cost-Effective Maintenance / Rehabilitation

The pavements in the network can then be categorized according to their condition. This is easily done with the aid of the PCI rating system. Every pavement may be placed into one of the following four categories.

Type I Pavements which have little or no load-related or nonload-related distress. A pavement in this category may be described as "excellent" or "very good." Its PCI is between 100 and 70.

Type II Pavements which have significant amounts of distress which is predominantly nonload related. A pavement in this category may be described as "good," "fair," or "poor." Its PCI is between 69 and 26, and more than 50 percent of its PCI deduct values are for nonload-related distresses.

Type III Pavements which have significant amounts of distress which is predominantly load related. A pavement in this category may be described as "good," "fair," or "poor." Its PCI is between 69 and 26, and more than 50 percent of its PCI deduct values are for load-related distresses.

Type IV Pavements which have extensive amounts of distress which may be load related or nonload related or both. A pavement in this category may be described as "very poor" or "failed." Its PCI is between 25 and 0.

Example of LCC Analysis Conducted for Cities in SF Bay Area.

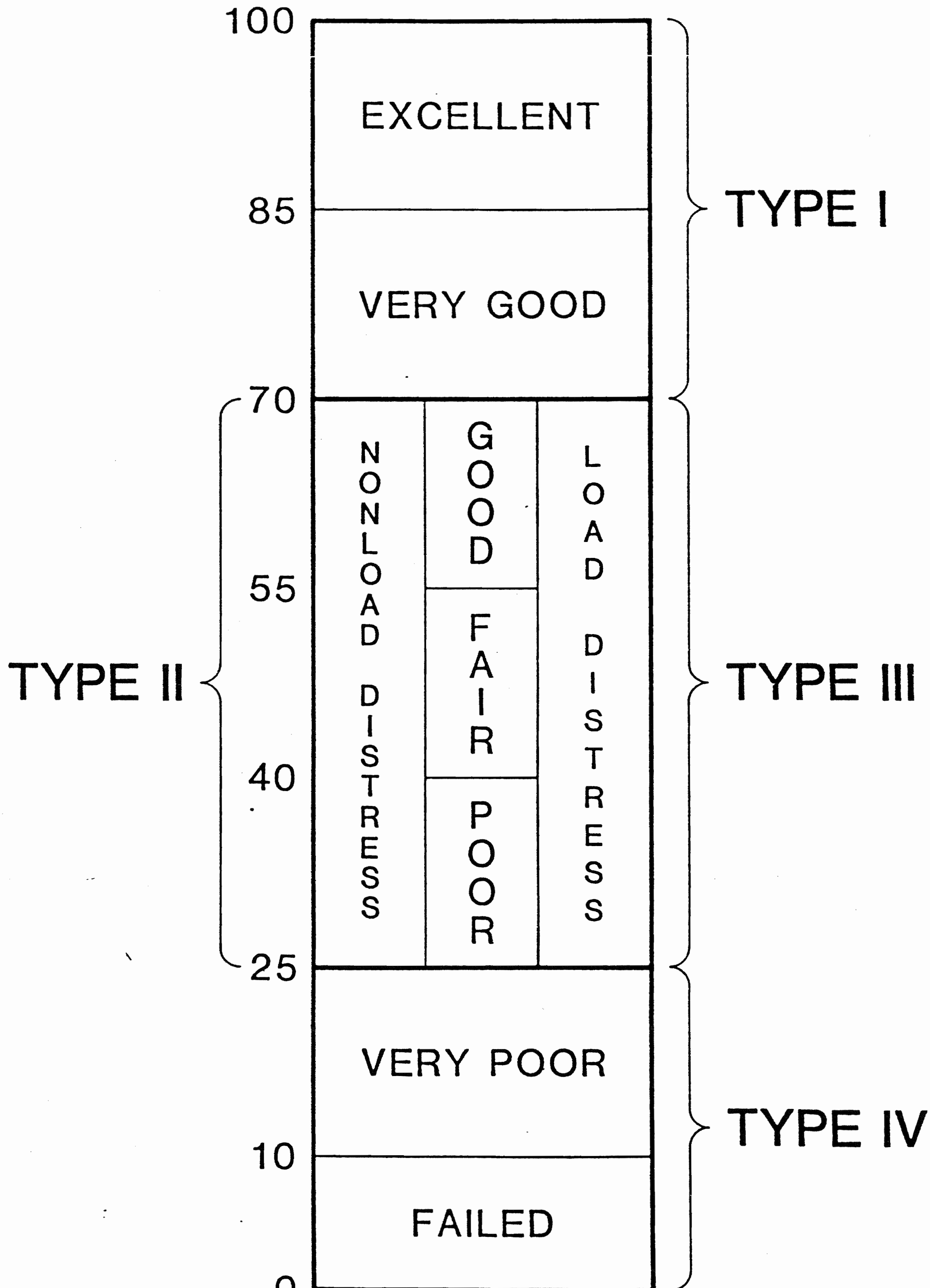

Figure 3.1 Categorizing Pavements by PCI

Table 3.4 Illustration of How to Compute the Equivalent Uniform Annual Cost (EUAC) of a Single Alternative (Cont'd)

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6. Average Annual Cost Per Square Yard (EUAC)

The Present Value can be converted to an average annual cost over the analysis period by multiplying it by the Capital Recovery Factor (CRF):

$$CRF = [int(1 + int)^n]/[(1 + int)^n - 1]$$

where int = interest rate
n = number of years

$CRF = \frac{i(1+i)^n}{(1+i)^n - 1}$

i = discount rate

$$EUAC = 6.75\ [0.10(1 + 0.10)^{20}]/[(1 + 0.10)^{20} - 1]$$
$$= \$0.79/sy$$

---

7. Actual Costs

The actual agency costs of performing the maintenance work given a 4 percent interest rate is:

Year	Actual Cost
1984	\$4.05/sy
1994	$4.05\ (1 + 0.04)^{10}$ = \$5.99/sy
1999	$0.91\ (1 + 0.04)^{15}$ + \$1.64/sy
	Total Actual Cost = \$11.68/sy

---

8. Summary

The total cost of performing this maintenance work over the next twenty years can be expressed in three ways:

Present Value = \$6.75/sy
Annual Cost = \$0.79/sy
Sum of Actual Inflated Costs = \$11.68/sy

Table 3.9 Average Annual Costs For Various Maintenance Treatment Strategies and Pavement Conditions For ARTERIAL Streets

No.	Maintenance Strategy	Future Maintenance Treatment	Pavement Condition I	II	III	IV
1A	Reconst.Surf.& Base	None	3.05	3.05	3.05	3.05
1B	Reconst.Surf.& Base	Chip Seal	2.86	2.86	2.86	2.86
1C	Reconst.Surf.& Base	Thin OL	2.84	2.84	2.84	2.84
2A	Reconst. Surface	Reconst.	0.89	0.89	1.10	1.15
2B	Reconst. Surface	D.Chip Seal	0.91	0.91	0.93	0.94
2C	Reconst. Surface	Thin OL	0.90	0.90	1.00	1.03
3A	Thick Overlay	Thin OL	0.66	0.76	0.94	1.15
4A	Thin Overlay	Thin OL	0.47	0.55	0.86	1.06
4B	Thin Overlay	D.Chip Seal	0.41	0.54	0.79	1.37
5A	Overlay w/Fabric	OL w/Fabric	0.45	0.60	0.75	0.94
5B	Overlay w/Fabric	D.Chip Seal	0.48	0.55	0.67	0.85
6A	SAMI & Thin OL	Thin OL	0.63	0.77	0.93	1.22
7A	Heater Scar.& OL	H/S & OL	0.63	0.71	0.95	1.10
9A	Single Chip Seal	Chip Seal	0.18	0.34	0.88	1.42
10A	Double Chip Seal	D. Chip S.	0.18	0.31	0.81	1.72
11A	Rubberized Seal	Rubber Seal	0.26	0.37	0.90	1.60
13A	Rejuv. Seal	Rejuv.Seal Chip S.,Thin OL	0.20	0.33	0.95	1.70
14A	Seal Cracks/Patch	S. Cks/Pat.	0.28	0.51	1.32	4.79

Notes: (1) All costs are in $/sy per year
(2) Interest = 10 percent
(3) Inflation = 4 percent

Table 3.10 Summary of Five Most Cost-Effective Strategies for ARTERIAL Streets

EXISTING PAVEMENT CONDITION			
I	II	III	IV
Single Chip Seal, Repeated (0.18)	Double Chip Seal, Repeated (0.31)	Overlay w/Fabric & Double Chip S. (0.67)	Ovly w/Fabric D. Ship S. (0.85)
Double Chip Seal Repeated (0.18)	Rejuv. Seal Chip S. (0.33)	Overlay w/Fabric Repeated (0.75)	Reconstruct Surf. & Double Seal (0.94)
Rejuv. Seal, Chip S., (0.20)	Chip Seal Repeated (0.34)	Thin Overlay & Double Seal (0.79)	Overlay w/ Fabric Repeated (0.94)
Rubber Seal Repeated (0.26)	Rubber Seal Repeated (0.37)	Double Chip Seal Repeated (0.81)	Reconst.Surf. & Thin OL Repeated (1.03)
Seal Cracks & Patch Repeated (0.28)	Seal Cracks & Patch Repeated (0.51)	Thin Overlay Repeated (0.86)	Thin Overlay Repeated (1.06)

*Equivalent Uniform Annual Cost in $/sy

Table 10.1 Salvage Value as a Percent of Initial Cost

Percent of Road Meeting Present Design Standards	0% to 25%			25% to 50%			50% to 75%			75% to 100%		
Analysis Period (years)	10	20	30	10	20	30	10	20	30	10	20	30
Type of Material												
1. Subbase (lower part of pavement structures)	30	25	20	55	50	45	80	75	70	95	90	85
2. Granular Base Material	20	15	10	40	35	30	60	55	50	80	75	70
3. Treated Base (with Asphalt, Lime, Cement)	25	10	0	35	20	10	45	30	20	55	40	30
4. Asphalt Surface or PCC	15	0	0	25	10	0	35	20	10	45	30	10
[redacted]	[redacted]			[redacted]			[redacted]			[redacted]		

Suggestion: These percentages may be revised up or down depending on the Engineer's best estimate of the value of the material at the end of the analysis period.

Texas Highway Dept.
Pavement Design Manual
1973

Table 3.6 Mean San Francisco Bay Area 1983 Unit Costs Used in Economic Analysis (Cities and Counties)

Maintenance Treatment	In-Place Unit Costs ($/sy)	
Reconstruct Surface and Base	26.00	Arterial
	23.00	Residential
Reconstruct Surface	7.60	Arterial
	6.00	Residential
Thick AC Overlay (2.5 inches)	5.42	Arterial
	4.50	Residential
Thin AC Overlay (1.5 inches)	2.78	All
AC Overlay w/Fabric	3.67	All
SAMI and Thin AC Overlay	4.50	All
Heater Scarify and Thin AC Overlay	4.70	All
Slurry Seal	0.62	All
Single Chip Seal	0.43	All
Double Chip Seal	0.78	All
Rubberized Chip Seal	1.50	All
Chip Seal and Slurry Seal	2.25	All
Seal Cracks	0.50	All
Rejuvinating Treatment	0.26	All

Notes: (1) Cost data was averaged from 1983 projects from ten local jurisdictions in San Francisco Bay Area.

(2) No patching costs are included. See Table 3.7 for estimates of patching costs.

Table 3.4 Illustration of How to Compute the Equivalent Uniform Annual Cost (EUAC) of a Single Alternative

1. Interest Rate = 10 percent
   Inflation Rate = 4 percent

2. Analysis Period = 20 years

3. Maintenance Strategy:

Thin OL & Patch (1984)	---------	Thin OL & Patch (1994)	-----------	Chip Seal Coat (1999)	-----------	End (2004)

4. Costs in 1984
   Thin OL & Patch = \$4.05/sy
   Chip Seal Coat & Patch = \$0.91/sy

5. Present Costs (1984)

$$\text{Present Value} = C_i\,[(1 + inf)/(1 + int)]^n$$

where C_i = cost at present year
inf = inflation rate
int = interest rate
n = number of years

Present Value of Thin OL & Patch: $= 4.05\,[(1 + 0.04)/(1 + 0.10)]^{10} = 2.31$

Present Value of Chip Seal & Patch: $= 0.91\,[(1 + 0.04)/(1 + 0.10)]^{15} = 0.39$

Maintenance	1984 Cost	Year Performed	Present Value
Thin OL & Patch	\$4.05/sy	1984	\$4.05/sy
Thin OL & Patch	&4.05/sy	1994	\$2.31/sy
Chip Seal & Patch	\$0.91/sy	1999	\$0.39/sy
		Total Present Value =	\$6.75/sy

ECONOMIC ANALYSIS OF ELEMENTS IN PAVEMENT DESIGN

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As competition for transportation investment dollars increases, all levels of management are being encouraged to become more cost conscious. This need for economy has resulted in haphazard implementation of cost reduction measures such as lowering pavement thickness standards or postponing construction. However, unless all implications of these measures are properly quantified in terms of trade-offs between present and future costs, these measures may, in fact, result in even higher overall pavement expenditures. A systems methodology to enable quantification of the trade-offs between various cost components of a pavement and selection of an optimum investment policy for any given situation is described. Through comprehensive analyses of alternative pavement strategies, the methodology provides individual cost components and the total cost of each alternative. Initial capital cost, resurfacing cost, maintenance cost, traffic delay cost during future resurfacings, salvage return at the end of the analysis period, and user costs are discussed, and it is demonstrated that certain elements, such as user costs, can be highly significant. An example is given to illustrate that trade-offs between various aspects of design as reflected by these costs can be efficiently studied by the methodology. Use of this methodology will enable agencies to develop uniform policies for cost reduction measures and alternative pavement standards.

•USE of economic analysis in highway engineering has received much attention over the years in an effort to provide highway authorities with better decision-making tools (1, 2, 3). The analysis has generally been applied to highway projects and even to highway networks, and the techniques are now being extended into pavement design (4, 5, 6, 7). In the course of seeking an improved decision-making tool for pavement management, it became apparent that direct agency costs of construction, rehabilitation, and annual maintenance did not provide a sufficient basis for determining the pavement structure design. The cost implications of lowered service to the public in terms of additional user operation costs, due to rougher pavements, and delay costs, due to traffic impedance during rehabilitation and maintenance, should also be included in the economic analysis. Management needs to find the middle ground to satisfy the objectives of providing an adequate service that satisfies the user but that keeps agency costs within imposed budget limitations.

A transportation agency that is responsible for providing and maintaining a system of roads that satisfies the present and future highway needs of a community is usually faced with budgetary constraints. The resulting situation for highway investment dollars generally leads the agency into haphazard implementation of cost reduction measures. The agency may be tempted to lower expenditures and accept the adverse implications of the additional user operation costs. However, this may not necessarily be in the best interests of the user; therefore, any such action must be preceded by an extensive economic evaluation that includes all the relevant agency and user cost implications. Such an evaluation methodology is presented in this paper. Its use in the

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formulation of policies will enhance an agency's credibility and increase public acceptance of its decisions by enabling the agency to demonstrate reasonableness and objectivity in its decision-making processes.

In contrast to bridge and building structures, pavement structures do not fail catastrophically. Instead, pavements slowly deteriorate in riding quality, safety, structural capacity, and structural integrity with traffic and time. As an example, the pavement performance with respect to riding quality and safety is shown in Figure 1. In this paper only the economic consequences of riding quality performance are considered. The other aspects of performance may be considered in a similar manner.

The economic evaluation methodology and its elements are described below.

ELEMENTS OF PAVEMENT COST ANALYSIS

There are six major elements of cost that must be evaluated for each pavement strategy:

1. Initial capital cost,
2. Future resurfacing costs,
3. Maintenance cost,
4. Traffic delay cost during future resurfacings,
5. Salvage return at the end of analysis period, and
6. User costs of vehicle operation (i.e., time, accident, and discomfort).

These cost elements do not include all the costs involved in a highway project; however, a pavement design analysis should include only those costs that are related to the pavement. For example, right-of-way width is generally decided in the planning stage of a highway and is not a function of pavement strategy; therefore, it should not be included in the economic analysis of pavement design.

The six cost elements will be described in detail below, but, to facilitate understanding, the following example is presented for use in showing the various cost computations associated with the analysis: A four-lane divided rural highway is required for a total service life of 30 years including future resurfacings. The annual average daily traffic (AADT) in the first year after construction is expected to be 10,000 vehicles per day, and the annual average growth will be 5 percent, resulting in 25,000 vehicles per day by the end of 30 years.

Initial Capital Cost

Initial capital cost involves first the selection of possible initial designs to cover a spectrum of variety and experience. Designs with layer thicknesses slightly smaller and larger than those generally used for similar situations should be included. Alternatively, for agencies that base their selection on certain allowable deflections, additional designs that result in slightly lower and higher deflections should be included.

In the example described above, five initial roadway designs have been selected for analysis. A description of the designs is given in Table 1.

The next step is to compute the initial construction cost for each design selected. This involves the calculation of quantities of materials for each pavement structure. Material quantities are functions of their thicknesses in the structure as well as thicknesses of other layers and the width of pavement and shoulders. The cost of in-place material in a pavement structure depends on the quantity to be provided, the construction procedure used, the length of the project, and many other factors.

For the example described above, full-width granular sections are assumed for the five designs, and initial capital costs are computed based on assumed material prices. The initial capital cost for each design is given in Table 2. If an agency's decision is based on only initial capital cost, design C with the lowest initial cost of $198,600/mile ($124,125/km) would be the obvious choice. However, it will be shown later that, when future expenditures for the five designs are also considered, design C proves to be a poor choice.

Figure 1. Pavement performance measured by riding comfort index and skid safety index.

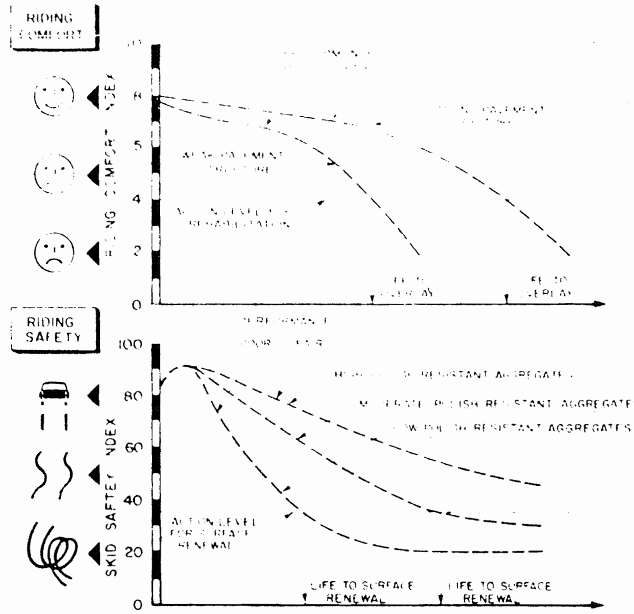


Table 1. Five initial roadway designs for example problem.

Design	Description	Actual Thickness (in.) Surface	Base	Subbase	Equivalent Granular Base Thickness (in.)
A	Conventional	5	10	9	26
B	Conventional	4	6	15	24
C	Conventional	6	6	6	22
D	Deep strength	8	12	–	28
E	Full depth	12.5	–	–	25

Note: 1 in = 25.4 mm. Layer equivalency factors: surface:base:subbase = 2:1:2/3.

Table 2. Various cost components for example designs.

Item	Design A	Design B	Design C	Design D	Design E
Initial capital cost, pavement plus shoulder	232,950	203,950	198,600	259,350	244,850
Resurfacing cost, overlay plus shoulder upgrading	49,300	70,450	81,300	27,300	52,750
Subtotal	282,250	274,400	279,900	286,650	297,600
Maintenance cost	35,100	31,400	27,500	41,400	34,300
Subtotal	317,350	305,800	307,400	328,050	331,900
Traffic delay cost	5,100	10,300	7,100	1,450	4,550
Subtotal	322,450	316,100	314,500	329,500	336,450
Salvage return values	13,550	15,100	16,300	13,100	17,200
Subtotal	308,900	301,000	298,200	316,400	319,250
Extra user cost	92,950	108,900	140,050	64,350	83,750
Total	401,850	409,900	438,250	380,750	403,000

Note: All costs are expressed as dollars per mile. 1 mile = 1.6 km.

Table 3. Various possible overlay thickness strategies for 2 to 4-in. (51 to 102-mm) thickness of one overlay.

No. of Strategy	Overlay Thickness (in.)	No. of Strategy	Overlay Thickness (in.)
1	2, 2, 2	·	·
2	3, 2, 2	·	·
3	4, 2, 2	·	·
4	2, 3, 2	18	4, 4, 3
5	3, 3, 2	19	2, 2, 4
6	4, 3, 2	20	3, 2, 4
7	2, 4, 2	·	·
8	3, 4, 2	·	·
9	4, 4, 2	·	·
10	2, 2, 3	17	4, 4, 4
11	3, 2, 3	·	·
		·	·
		·	·

Note: 1 in = 25.4 mm.

Resurfacing Cost

Resurfacing cost includes future overlays or upgradings made necessary when the riding quality, or riding comfort index (RCI), of a pavement reaches a certain minimum level of acceptability. The riding quality of a pavement is described by the RCI in Canada and the present serviceability index (PSI) in the United States. The minimum acceptable level of riding quality generally depends on the function and classification of the highway. The maximum value of RCI and PSI is usually obtained immediately after initial construction and depends on construction quality, type of aggregate in the surface layer, material quality control, construction tolerances, and many other factors.

Essential to the determination of resurfacing costs are the algorithms that predict the number of years at which a pavement reaches the minimum specified level of roughness after initial or overlay construction. The models necessary to predict the performance histories of pavement structures are still under development by various agencies. Ontario has developed a performance prediction model based on principles of linear elastic theory combined with AASHTO and Brampton Road Test data and general experience (8). For those agencies where such prediction models are not available, objective information acquired to date regarding pavement lives, and subjective judgment based on local experience, can be used in the interim to estimate lives.

Although a pavement designer may in some cases be able to recommend the type and thickness of a resurfacing based on past experience, accurate prediction of overall optimal strategy consisting of more than one overlay during a pavement's life is extremely difficult. For example, if 2 to 4-in.-thick (51 to 102-mm) are allowed and if algorithms are available to predict the lives of overlay thicknesses, there can be many combinations of one or more overlay thicknesses to obtain the desired total service life of a highway. An example of all possible overlay thickness strategies for this case is given in Table 3.

A review of the overlay strategies in Table 3 may cause one to question the reasons for analyzing so many strategies of overlay construction. For example, if three 2-in. (51-mm) overlays (strategy 1) successfully give a certain total service life for a pavement, why is it necessary to investigate strategy 2, which is similar to strategy 1 except for an extra inch (millimeter) of thickness in the first overlay? First, it is possible that an extra inch (millimeter) of the initial overlay may extend the life of the pavement so that only one additional 2-in. (51-mm) overlay may be needed to obtain the desired total life. Second, even if both strategies require three overlays, the extra 1-in. (25.4-mm) overlay in strategy 2 will increase the time until the next two overlays are needed in this design strategy and thus reduce the present values of these overlays. Such a reduction in the present values of these overlays might offset the additional cost (in terms of present value) of providing 1 in. (25.4 mm) of extra thickness of the first overlay. This might cause strategy 2 to be more economical than strategy 1.

For the five example designs (Table 1), approximate initial lives have been predicted, and example overlay policies have been selected. Costs of overlays, their discounted values, and subtotals of initial construction and overlay costs are given in Table 2 for each of these designs. Design C, an optimal design based on initial capital cost alone, is no longer the most economical design after overlay costs are added. Instead, design B becomes the most economical.

Maintenance Cost

A comprehensive economic analysis should include the estimation of all costs that are essential to maintain pavement investment at a desirable level of service or at a specified rate of deteriorating service. The level of maintenance, i.e., the type and extent of maintenance operations, affects the rate of loss of the RCI. Alternatively, if a certain performance history is desired, a specific level of maintenance will be necessary.

Performance as a function of maintenance level is shown in Figure 2. A maintenance level M_1 results in a performance history P_1 for a particular design. When the maintenance level is increased to M_2 the resulting performance history is P_2. The dditional maintenance expenditure of $M_2 - M_1$ has bought an additional life of $t_2 - t_1$ years. Note, in Figure 2a, however, that as time passes relatively larger increments of maintenance cost have to be expended to buy every increment of additional life.

As shown in Figure 2, a maintenance level can be represented by an annually increasing cost curve. The maintenance cost is minimal in the first year after initial construction, resurfacing, or reconstruction and gradually increases at a progressive rate.

Various maintenance operations such as pavement and shoulder maintenance, drainage and erosion, vegetation, structures, and snow and ice control are carried out for a highway. However, for pavement economic analysis, consideration should be given to only those items of maintenance that directly affect pavement performance, such as maintenance of pavement surface and shoulders. Of course, if any other item of maintenance affects pavement performance and if the trade-off between the cost of this maintenance item and the pavement performance can be quantified, this cost may also be included in the economic analysis. Maintenance costs of five example designs are given in Table 2.

Traffic Delay Cost

Overlay construction generally disrupts traffic flow and causes vehicle speed fluctuations, stops and starts, and time losses. The extra user cost thus incurred is often a significant proportion of the total overlay cost and may warrant its inclusion in the economic analysis. This indirect, nonagency cost has never in the past been given due consideration and has traditionally been considered only as a soft cost; however, the extra user cost is an expense to the road users and, therefore, should be included in the economic analysis.

Traffic delay cost can be defined as a function of traffic volume, road geometrics, time and duration of overlay construction, road geometrics in the overlay zone, and traffic diversion method used. Cost is comprised of user time and vehicle operating values resulting from driving slowly, fluctuating speeds, stopping, accelerating, and idling. Based on traffic demand and available road capacity at the time of overlay construction, traffic delay costs can result from either of the following two situations:

1. Vehicle slows down to overlay zone speed, continues at this reduced speed through overlay zone, and accelerates back to the original speed; or
2. Vehicle stops, idles for a certain time, accelerates to overlay zone speed, continues at this speed through the overlay zone, and accelerates back to the original speed.

The latter situation develops when traffic volume at any time during overlay construction exceeds the available capacity; the former situation is more prevalent and generally occurs when traffic volumes are low and, as such, do not cause traffic stopping and idling. Traffic delay cost as a function of traffic volume is shown in Figure 3. The cost gradually increases as a function of AADT to a point where traffic volume in the peak hour is still smaller than available capacity through the overlay zone. AADT, which exceeds this value, results in vehicle stopping and idling and thus in a sharp increase in traffic delay cost. When traffic volumes are in this sharply increasing range, an overlay construction may cause such a significant traffic delay cost that the decision to provide an overlay may be changed. For example, if high traffic volumes are expected during the latter years of a pavement's life, optimum pavement design may favor stronger initial pavement or thicker overlay construction during the early years of a pavement's life.

Traffic delay costs for five example designs are given in Table 2. These are the discounted values of tràffic delay costs during all overlays for each design strategy. When added to the sum of all previous costs, the subtotals show a shift of the optimal design among the five designs. Design C now becomes the optimal rather than design B, which had the least cost when only initial, overlay, and maintenance costs were considered.

Figure 2. Performance as function of maintenance level.

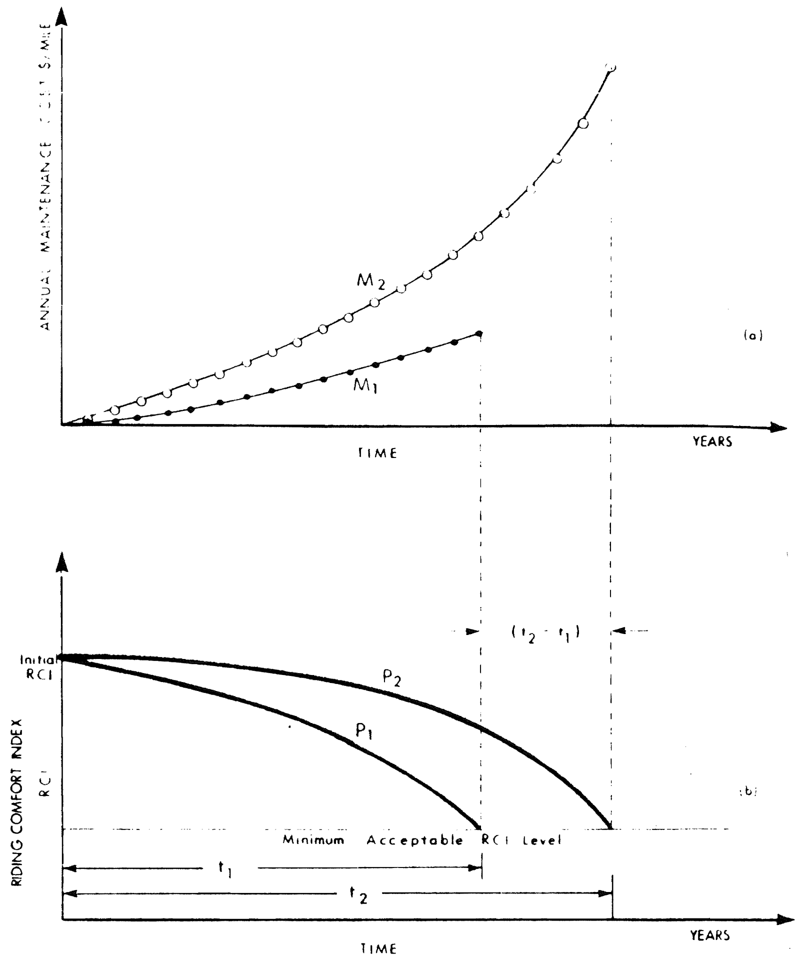
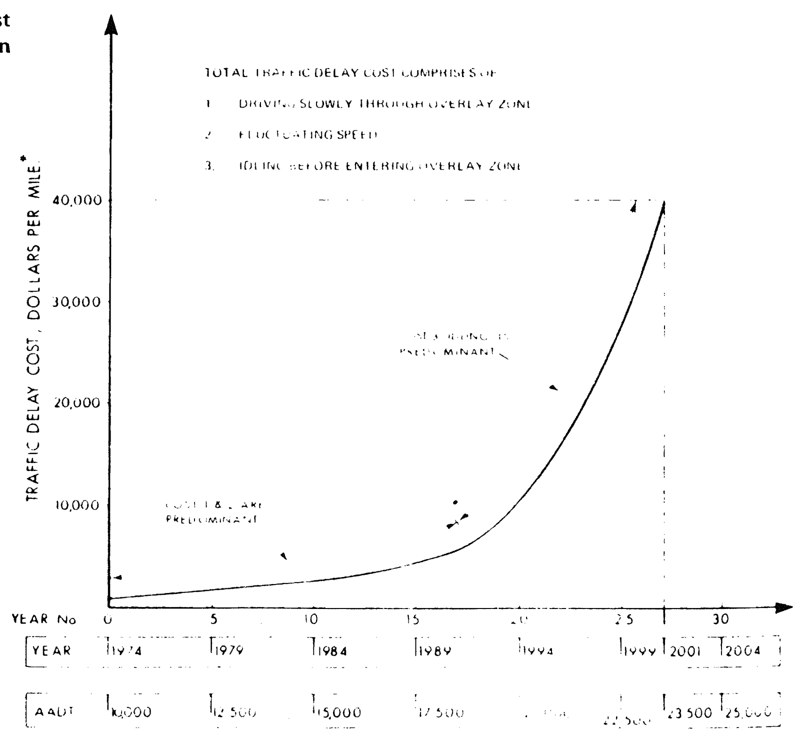


Figure 3. Traffic delay cost during overlay construction in various years in future.



Salvage Return

Salvage return of a strategy is the value of a pavement at the end of its analysis period. Computation of this cost allows for a rational comparison of designs that have different material quantities and that are at different levels of roughness at the end analysis period. Although the economic comparison of various strategies is conducted over a fixed analysis period, actual useful service life of a design generally extends beyond this period. This extension, as shown in Figure 4, may be of different magnitude for every design and, therefore, should be taken into account in any economic comparison. For example, if designs B and C in Figure 4 are compared, the economic value of design B after 30 years will be higher than that of design C since the former is overlaid in the twenty-seventh year (longer residual life) and the latter is overlaid in the twenty-third year.

Salvage value of a material is a function of several variables. In addition to the volume of such material, it depends on when the material was provided, its durability, its position with respect to other pavement layers, and its anticipated use at the end of its service life. Salvage value of a material may be defined as a percentage of the original cost of the material. It should be emphasized, however, that percent salvage return can only be applied to that part of total material cost resulting from unit material price and not to the incremental unit price resulting from labor cost. Salvage percentage can be a negative value if it is anticipated that the material will have no use at the end of its service life and will have to be hauled away at extra cost.

Determination of the salvage value of the number of years that a strategy extends beyond the required service life is relatively difficult. However, a relationship between the salvage value of a strategy and its extended life can be developed.

Salvage return (a negative cost) should be discounted from the end of the analysis period (Table 2).

User Cost

Each alternative pavement design is associated with a number of indirect (soft) costs that accrue to the road user and must be included in a rational economic analysis. Similar to pavement costs, user costs are related to the performance history of the pavement. A pavement design that provides an overall high level of roughness over a longer time period will result in a higher user cost than a design that provides a relatively smooth surface for most of the time.

The four major types of user costs associated with a pavement's performance are as follows:

1. Vehicle operating cost consisting of fuel consumption, tire wear, vehicle maintenance, oil consumption, vehicle depreciation, and parts replacement;
2. User travel time cost;
3. Accident cost consisting of fatal accidents, nonfatal accidents, and property damage; and
4. Discomfort cost.

Each of the above costs is a function of roughness level and of the resulting vehicle speed.

As a pavement becomes rough, the operating speeds of vehicles are reduced. Lower speeds and rough pavements increase traveling time, level of discomfort, and other user costs. Since level of roughness for a pavement design depends on its initial construction thicknesses and materials, the extent of rehabilitation, and the extent of major and minor maintenance provided during its service life, user cost is interrelated with all of these factors.

There is a lack of extensive data on user costs as related to a pavement's riding quality. This is partly because of the inadequate attention given to this aspect of highway transportation costs. However, some data are available (3, 11) for use in determining the user cost component.

Figure 4. Performance histories for example designs.

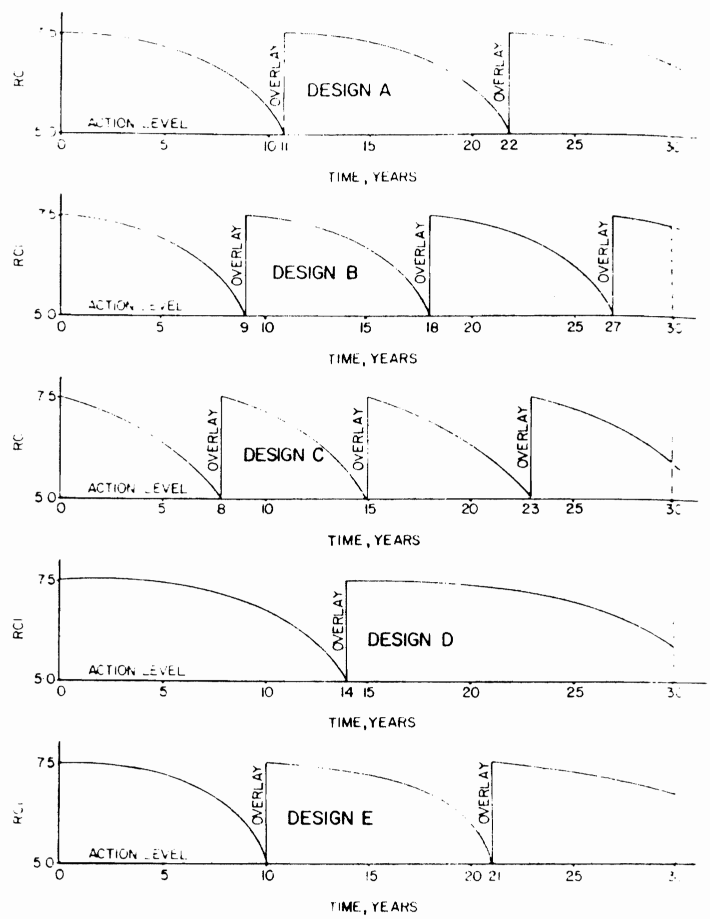
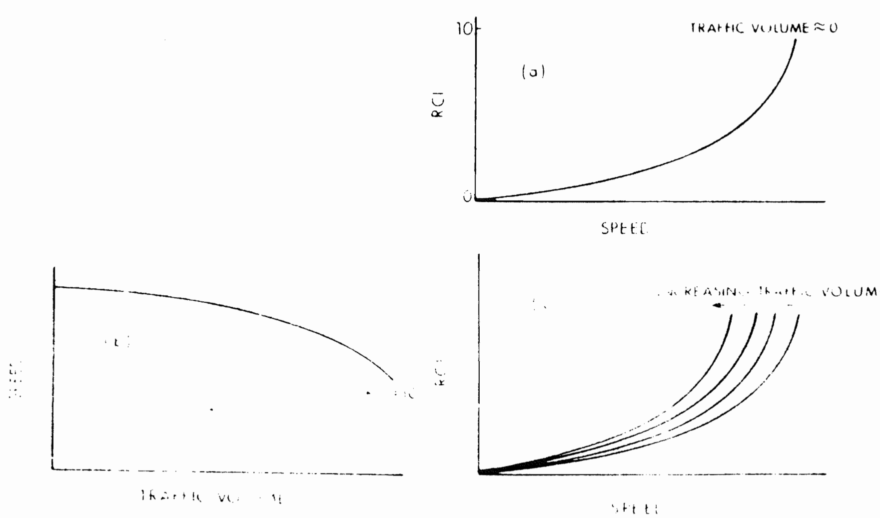


Figure 5. Speed as a function of roughness (RCI) and traffic volume.



Information necessary to determine total user cost on a pavement strategy falls into the following four major categories:

1. Determination of actual traffic expected on the facility and its anticipated growth during the service life;
2. Prediction of performance history of a pavement strategy in terms of its RCI versus age;
3. Determination of speed profiles adopted by motorists while driving on the particular pavement during its service life; and
4. Determination of user cost data for vehicle operation, travel time, and accident and discomfort, as functions of vehicle speed and the RCI.

Determination of speed profiles during the life of a pavement strategy is extremely important since it provides the means to relate user cost to the performance history. As the RCI of a pavement decreases with time, road users reduce vehicle speeds to adjust to the roughness of the highway. In addition to this adjustment, the driver may have to further reduce vehicle speed to adjust to the traffic volume. A typical roughness versus speed curve is shown in Figure 5a, in which the effect of traffic volume has been assumed to be negligible. Similarly, a typical traffic (congestion) versus speed curve is shown in Figure 5b, in which the effect of pavement roughness has been assumed to be negligible. Congestion is generally represented by a volume to highway capacity ratio (V/C), but in Figure 5b the term traffic volume has been used for simplification. Actual highway speed is a function of both roughness and traffic (congestion) as shown by curves in Figure 5c. The combined effect is not the simple addition of the two individual effects shown in Figures 5a and 5b, nor is it the minimum of the two. Although Figure 5c gives actual speed reduction on a highway, only a part of this reduction is attributable to pavement roughness and, therefore, to pavement design. The rest of the reduction results from capacity restrictions and, therefore, should be attributed to major reconstruction such as pavement widening, addition of lanes, and widening of shoulders.

Based on the simple roughness versus speed relationships shown in Figure 5c, Figure 6 shows speed profiles for two example designs. Speed is shown as a step function determined for the average RCI for each year. A lower overall average speed during the 30-year analysis period is observed on design T_1, which gives a relatively rougher surface when compared to design T_2.

A major data requirement for user cost computations is determination of unit costs [dollars per vehicle mile (kilometer) of operating, travel time, accidents and discomfort] as functions of speed and pavement roughness (RCI). These unit costs, along with anticipated future passenger and commercial traffic, lead to the calculation of total user cost on a pavement strategy.

Figure 7 shows typical unit cost curves as functions of operating speed and pavement roughness for each category.

For pavement economics, user cost should be represented by a difference between total user cost of traveling on a rough pavement and the total user cost of traveling on the smooth pavement had it stayed at the RCI level at which it was initially constructed. This difference is the extra user cost. Given the performance history of a pavement, its average RCI for each year determines the average operating speed for that year, and this in turn gives the extra user cost. Extra user cost per year, when discounted to its present value and summed over the service life of a pavement, gives the total extra operating cost for a strategy. The addition of extra operating costs to the subtotal of all previous costs (Table 2) shifts the optimal strategy from design C to design D.

SELECTION OF DESIGN

To achieve maximum economy for a pavement design, one should analyze a large array of alternatives. The alternatives should include all the available materials, various combinations of their thicknesses, and various policies of maintenance and resurfacings.

Figure 6. Speed profiles for example designs T_1 and T_2.

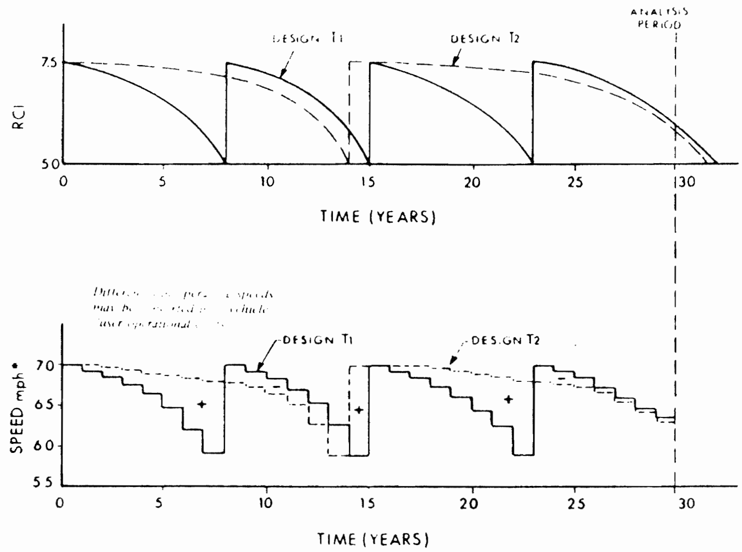
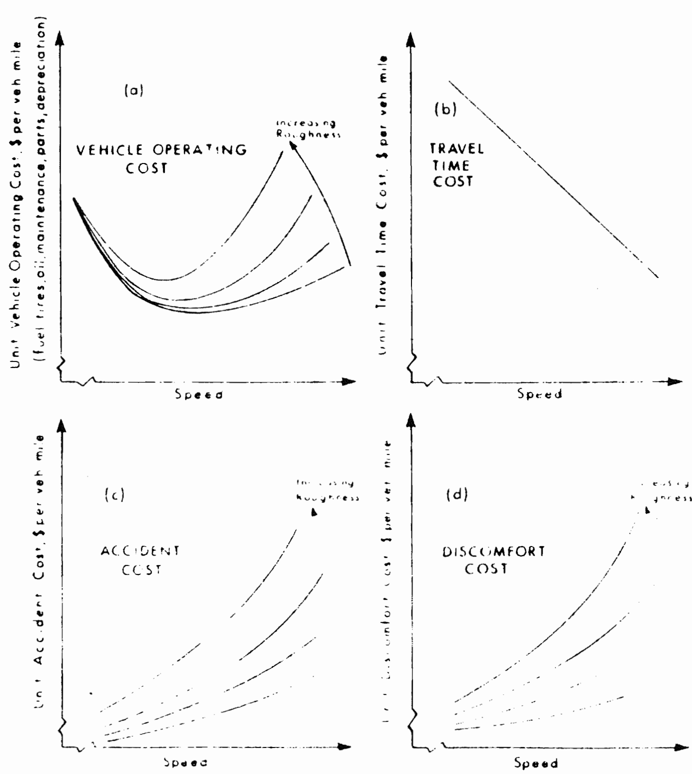


Figure 7. Trends of user cost components as function of operating speed and pavement roughness.



Cost analyses like those in the previous sections should then be conducted, and the design with the least present worth of total overall cost should be selected.

Implications

It is difficult to quantify the relative importance that the decision maker would ascribe to various economic, social, and experience values based on current knowledge. The administrator, therefore, should be presented with design and cost details of optimal and several nearly optimal designs and should then select a design that is economically justified and in accordance with past experience. Cost analysis will provide information about the magnitudes of various cost components.

The role of cost analysis, using cost computations of the five example designs, in the selection of an optimal design strategy is shown in Figure 8. The effect of the addition of each cost component on the optimum solution and the least cost for the five example designs after each addition are also shown in Figure 8. Design C is the least costly design if only initial capital cost is considered; however, addition of resurfacing cost shifts the optimal to design B. Design C again results as a least costly design when maintenance and traffic delay costs are added; however, design D becomes the optimal when all the costs are included. This shift in the optimal design emphasizes why all significant cost components must be considered in pavement design. A design selected on the basis of initial capital cost alone or even with the addition of resurfacing cost may still not be the optimal design from the point of view of overall cost.

Additional Cost Components

Only six cost components have been discussed for use in the selection of a pavement design; however, there are other costs that may also be included to improve a rational pavement design decision.

A typical example is drainage and the associated maintenance. It is well known that improved drainage affects the performance of a pavement. However, it would not be rational to take into account the cost of drainage until the trade-off between this cost and pavement performance is defined in the form of an algorithm. When such an algorithm becomes available, alternate drainage designs can then be applied to each pavement design.

This leads to the definition of all costs that should be considered in an economic analysis of pavements. A simple definition is all costs that are functions of pavement layer thicknesses or that affect pavement performance and result in extra life or smoother pavement surface. As discussed above, algorithms to define trade-offs between cost and pavement performance must be established before such a cost is considered in economic analysis.

Cost components to be included in a pavement design decision also depend on the objectives and budget of the agency concerned. For some agencies, a design may be selected on the basis of only the costs that form their actual spending (agency costs). In such cases, traffic delay cost, extra user cost, and sometimes even salvage return of the pavement strategy may be ignored. For the example shown in Figure 8, design B would be considered optimal in such a case. However, even if cost components such as extra user and traffic delay cost are not included in the design selection process, these costs should still be computed and made available to the administrator as an aid in the decision-making process. Extra user and traffic delay costs will provide information about public acceptance of a design since these costs are good indicators of public reaction to an agency's design decision.

Significance of a cost component relative to total overall cost is another criterion that can be used to establish whether a cost component should be included in pavement economic analysis. Table 4 gives the relative significance of the six cost components that were analyzed for the five example designs (Table 1). Each cost is shown as a per-

Figure 8. Cost components for five example designs.

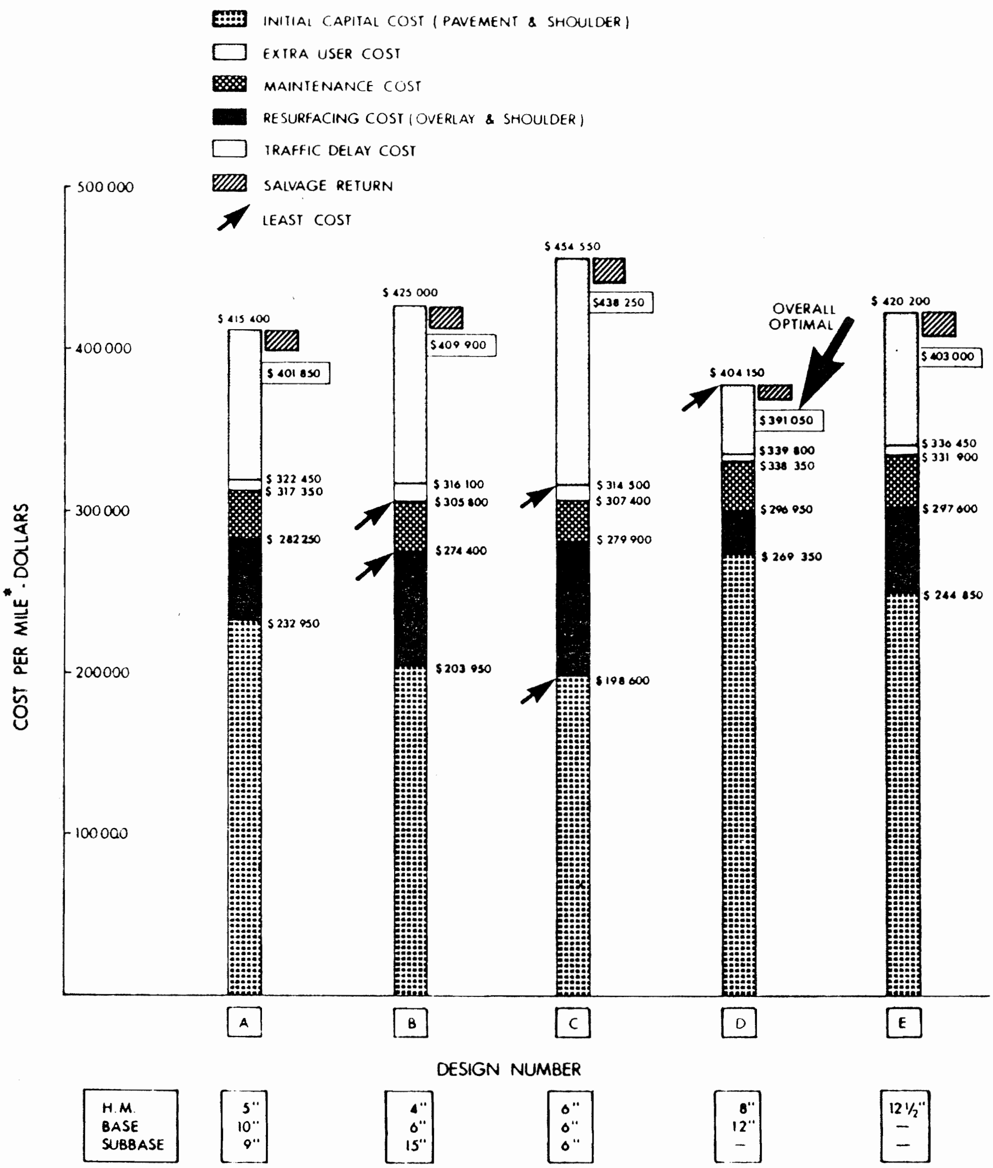


Table 4. Percentages of different cost components of five example designs.

Design	Initial Capital Cost	Resurfacing Cost	Maintenance Cost	Traffic Delay Cost	Salvage Return Value	Extra User Cost
A	58.0	12.3	8.7	1.3	3.4	23.1
B	49.8	17.2	7.7	2.5	3.7	26.6
C	45.3	16.6	6.3	1.6	3.7	32.0
D	66.3	[illegible]	10.6	0.4	3.3	16.5
E	60.8	13.1	8.5	1.1	4.3	20.8
Average	56.0	[illegible]	8.4	1.4	3.7	23.8

centage of total overall cost. A general review of Table 4 gives various cost components in their order of relative significance as follows:

1. Initial capital cost,
2. Extra user cost,
3. Resurfacing cost,
4. Maintenance cost,
5. Salvage return value, and
6. Traffic delay cost.

The relative significance of the various cost components is not constant and depends largely on the specific conditions of each design situation. For example, in the case of facilities carrying a high AADT, traffic delay cost may become a significant portion of total overall cost. Similarly, if the interest rate is lowered, the relative significance of all future costs may become greater as compared with the initial capital cost.

FUTURE RESEARCH

A comprehensive economic analysis approach for pavements has been described. A large number of algorithms to predict pavement and traffic behavior and to compute various cost components are required to conduct the analysis. Some of the available algorithms have been used for values in Table 2; others unavailable at present have been assumed. Research is required in the following major areas to develop new algorithms and data or to validate those presently available:

1. Prediction of performance histories for new as well as overlaid pavements,
2. Maintenance cost versus pavement performance prediction,
3. Traffic delay modeling and cost prediction,
4. Salvage value modeling,
5. Roughness versus speed relationships,
6. Future traffic predictions,
7. User cost modeling, and
8. Optimization modeling.

CONCLUSIONS

The economic analysis approach presented in this paper can be used to quantify trade-offs among various aspects of pavement design such as initial construction, resurfacing, maintenance, and user costs. A study of these trade-offs and optimization between various aspects of pavement investment will lead to economically sound decisions in pavement design.

The Ontario Ministry of Transportation and Communications has developed a computerized version of the economic analysis and optimization as discussed in this paper. The system, the Ontario pavement analysis of costs, is being used at present by the ministry to design pavements in the province.

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Economics of seal coating

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PAVEMENT CHIP SEALING is not new. In fact, a patent was taken out in June of 1873 by Abbott for a composition of tar and creosote spread hot with a mop or broom and covered with a layer of grit passing the 1/4-inch sieve. We can be certain that other concoctions have been used to seal pavement before and since Abbott's patent.

Asphalt cement seal coats were introduced in 1902 by the Warren Brothers. The bituminous cement was applied by squeegees, followed by two applications of hot aggregate, then steam rollers were used. Portable melting kettles were used to transfer the hot oil to barrels and then squeegeed on.

In 1900, horse-drawn distributors entered in, using gravity spigots or pipes with drill holes. Traffic then spread the asphalt over the surface because the application method caused streaking. Later a splash board was used to stop streaking. Then about 1910 the steam roller was used to apply pressure to distributor bar or nozzles.

The greatest advancement in seal coats developed in the 1930s when the truck-mounted pressure distributor was used with a variety of spreader boxes and end gate feeders for aggregate spreading.

Of course the primary requirement for a seal in the past, as well as today, is to apply one or more successive coats of asphalt binder and cover aggregate to a surface. The object is to provide a low-cost, all-weather, waterproof, skid resistant surface; give new life to dry, weathered surfaces; reinforce pavement; guide traffic; and improve visibility.

To do one or all of these, a design is recommended. This can be accomplished by methods established by Dr. Norman McLeod of The Asphalt Institute, and others. Considerations in a seal coat design are strength of foundation, materials (aggregate, asphalt), traffic, weather or climatic conditions, construction method, and experience.

Today there are several methods and combinations of methods for applying seal coats. Some of the more common methods are: 1) liquid asphalt and uncoated aggregate; 2) liquid asphalt and coated aggregate; 3) hot asphalt cement and uncoated aggregate; 4) hot asphalt cement and coated aggregate; 5) emulsified asphalt and uncoated aggregate; 6) emusified asphalt and coated aggregate; 7) slurry seal; 8) asphalt rubber and aggregate; 9) coal tars and aggregate; and 10) open-graded plant mix or popcorn seal. The choice of method is influenced by conditions such as aggregate quality, asphalt, climate, traffic, money, and the intended result.

Phoenix has been chip-sealing its streets for more than 40 years. There were less than 300 miles of streets in Phoenix then; today there are more than 3,500 miles. Chip sealing is the least expensive preventive maintenance program available. Phoenix chip seals about 300 miles annually. Chip sealing city streets is not always popular because of flying rock, asphalt pick-up, and noise, but even with these problems, there are many reasons why chip seal is used. Asphaltic concrete overlays cannot always be used because of set curb and gutter grades, and recycling is not always effective. Thin asphaltic concrete overlays crack quickly and cost about three to six times as much as chip seal. Chip seals reduce pavement temperatures and last longer — about 8 years on major streets and about 10 on residential.

With money enough in our maintenance budget to chip seal 300 miles annually, streets would receive a chip seal every 11.6 years. If the same maintenance money were to be placed into overlays, only 100 miles could be done annually. Overlays, normally good for about 10 years, would cost three times as much, and Phoenix could never maintain or retain its investment.

Phoenix uses several other preventive maintenance and maintenance methods under a management system. Asphalt rubber, introduced by Phoenix in 1969, is very successful, having lasted 14 years so far. It costs two to three times as much as standard chip seals, however. Slurry seal was first used this past year and is showing promise. Emulsified asphalt as a binder is being used now, but with limited success because of traffic problems. Grinding the pavement and overlaying works very well; however, the cost is four times that of the chip seal. Stress-absorbing membrane interlayers are very successful. This makes use of asphalt rubber or fabrics as interlayer with an overlay. It is used when there are very special problems. Because all these methods have advantages under given situations, we cannot afford to tie up maintenance money to one system. We must be flexible and creative.

The economics of chip seal in the 1980s make sense. Standard chip seals, using hot asphaltic cement, cost about 57 cents per square yard and last about 8 years, figuring out to about 7.13 cents per square yard per year. Asphaltic concrete costs about $1.49 per square yard per inch, with a life expectancy of 10 years, making it about 14.9 cents per square yard per year. Slurry seals are about 75 cents per square yard with a life expectancy of 5 years, adding up to about 15 cents per square yard per year.

Phoenix's success in seal coats is due primarily to choice of asphalt, selection of aggregate gradation, and method of application. We use AR-8000 or AC-40 asphalt cement for major streets and AR-4000 or AC-20 for residential streets. The aggregate is 3/8 inch nominal for major streets and 1/4 inch nominal for residential. This is good-quality, single-size aggregate. It is heated and pre-coated with less than 0.50% asphalt. Viscosity of the asphalt is temperature controlled. Boot trucks are certified and tested. Drivers are experienced and accurate. The chips are applied immediately behind the asphalt spreader truck and rolled with three pneumatic rollers. The construction train is not more than 1000 feet long.

Specifications are clear to do a good job. We have an unwritten requirement that is controlled by the inspector — that is to imbed 50% of the aggregate particle into the asphalt before the job is done. This can be accomplished through temperature control, application rates, application time, rolling pattern, and traffic.

The next day the loose chips are swept up, and the surface is examined to see that 50% are imbedded. If not, the new chip-sealed surface is fogged as needed with a diluted emulsion. The travelling public is not kept off the project except when absolutely necessary, however, it is asked to slow to 25mph until the loose chips are swept up. The finished product meets specifications with the least amount of incovenience at the lowest cost to the user. Chip seals are not new, however, they *have* improved. □

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 ($^1\!/_2$ to $^3\!/_4$ 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.

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 1. Typical partial and full transverse cracking.

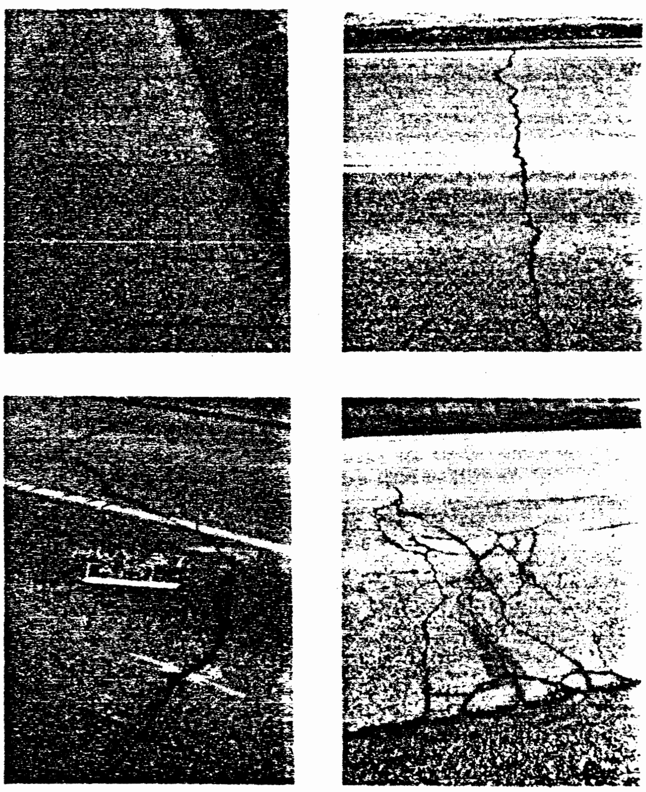
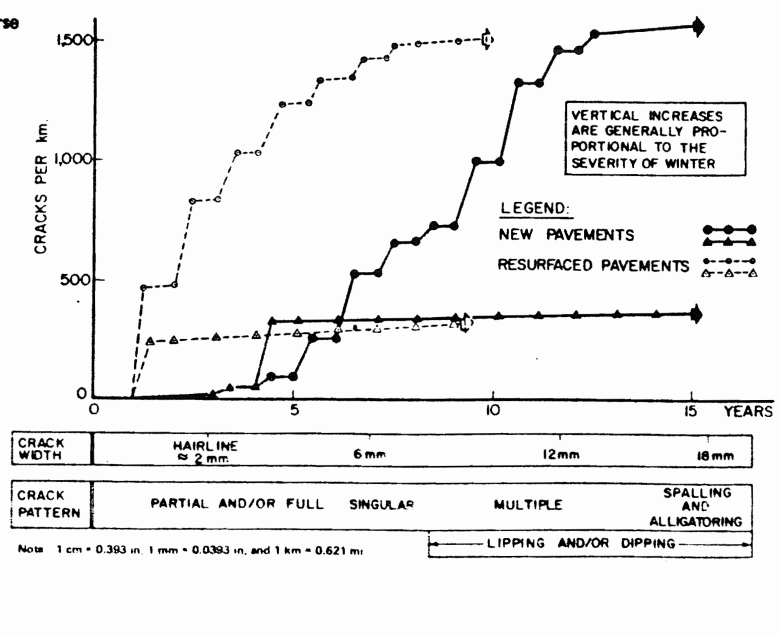


Figure 2. Schematic representation of transverse cracking in new and resurfaced pavements.



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.

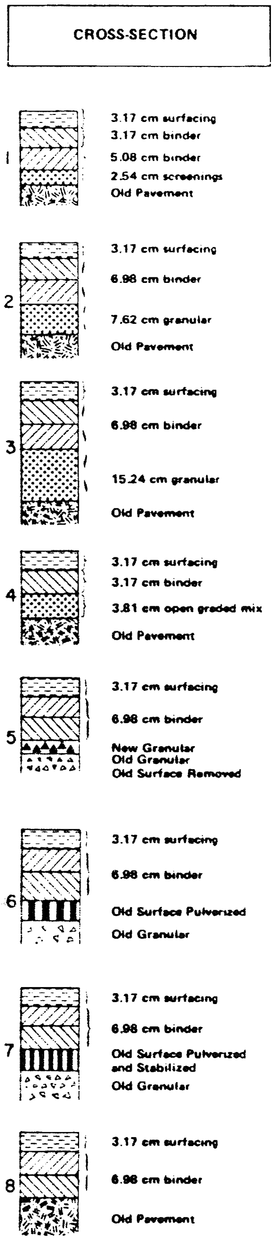
Cost

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.



CRACKS PER km

*W W W W W

Section	TYPE	MAY 72	NOV 72	MAY 73	NOV 73	MAY 74	OCT 74	MAY 75	NOV 75	JUL 76
1	**T	0	0	0	0	6	6	6	6	131
	**L	0	0	0	0	0	0	0	0	0
2	T	0	0	5	6	9	9	9	9	66
	L	0	0	1	1	2	2	2	2	2
3	T	2	2	5	5	8	10	10	10	47
	L	0	0	0	0	0	0	0	0	2
4	T	45	46	75	78	90	93	99	99	149
	L	0	0	0	0	7	9	21	21	34
5	T	1	1	3	3	7	7	8	8	19
	L	0	0	0	0	1	1	2	2	3
6	T	0	0	0	0	2	2	2	2	7
	L	1	1	1	1	2	2	2	2	2
7	T	0	0	0	0	0	0	0	0	5
	L	0	0	0	0	0	0	0	0	0
8	T	53	57	85	87	107	107	116	116	183
	L	0	0	0	0	3	3	17	17	42

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* (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.

*Based on 1971 bid prices for the test road sections.

Figure 4. Schematic representation of agency cost components.

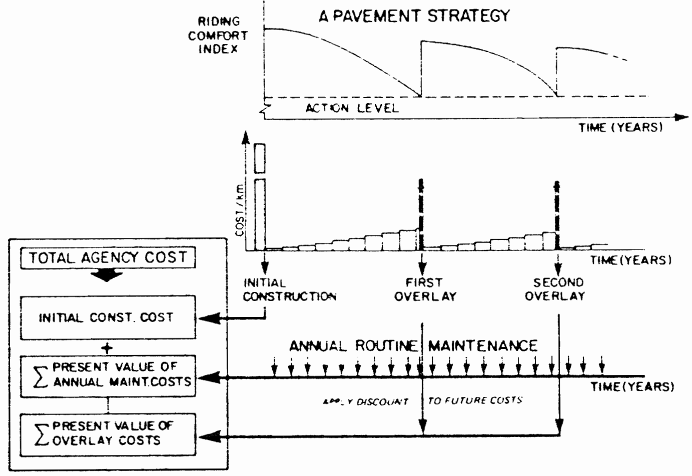


Table 2. Maximum prices for extra life with distortion correction.

Extra Life* (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	1.86	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.
*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_r), the treatment may be considered a viable alternative. The ratio of Δ_r 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_o) and future cost (R_f). Calculate for the treatment by using initial cost (C_o) and future costs (C_1, C_2, ... C_i). The extra initial cost ($\Delta_c = C_o - R_o$) and the future cost savings (Δ_r) respectively are as follows:

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

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_i, 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) × 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. $/m²)	Average Price (U.S. $/m²)	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 m² = 10.8 ft².

Figure 5. Three pulverization contracts in Ontario.

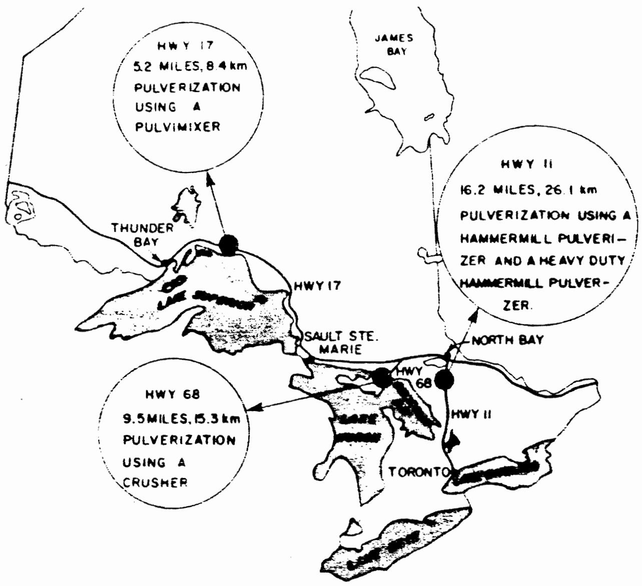


Figure 6. Pulverization by using a crusher.

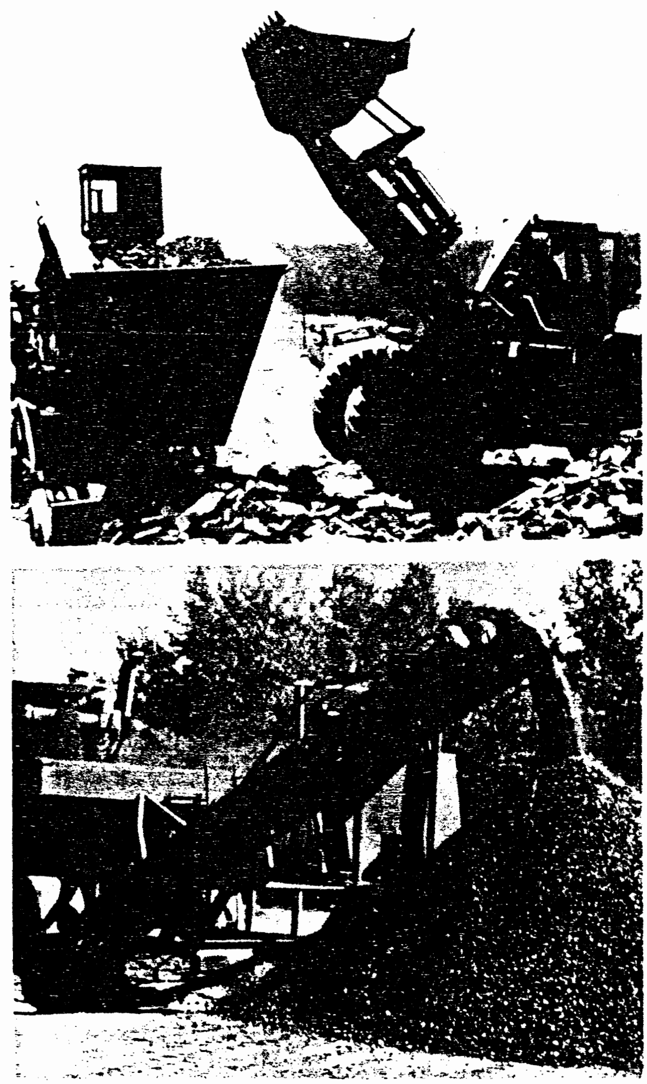
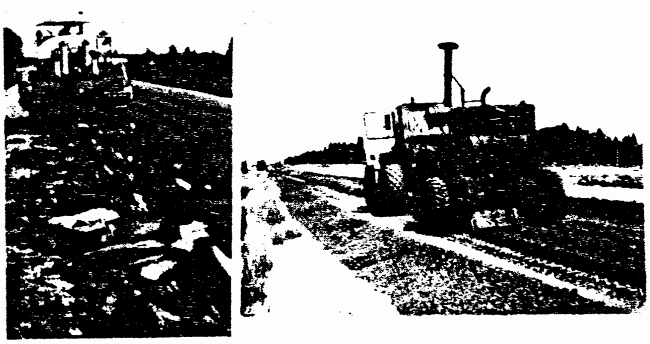


Figure 7. Pulverization by using a Hammermill pulverizer.



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.

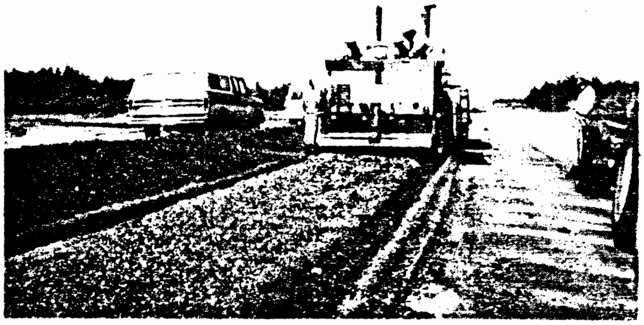
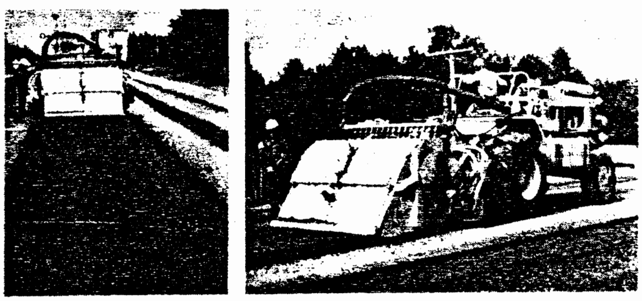


Figure 9. Pulverization by using a pulvimixer.



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 éffect 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,