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Pavement Management and Rehabilitation of Portland Cement Concrete Pavements

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Economic Analyses and Dynamic Programming in Resurfacing Project Selection

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The objective of this paper was to develop a dynamic-programming procedure by using economic analyses to assist in optimizing expenditures in pavement-resurfacing programs. Benefit relationships were determined from expected accident reduction, improved comfort, and savings in time, fuel, and maintenance. The only cost input to the program was the resurfacing cost of each project. Dynamic programming was adapted to the selection of projects for resurfacing in Kentucky. More than $8.4 million of additional user benefits would have been realized in 1976 if dynamic programming had been used in selecting projects. The benefit/cost ratio of sections selected for resurfacing by the current procedures was 3.21 compared with one of 4.22 if dynamic programming had been used.

Various management procedures and strategies may be employed to select and rank pavements for resurfacing. Subjective visual evaluations and objective measurements may be used alone or in combination. Sophisticated methods consider pavement roughness, skid resistance, traffic volume, and accidents in an economic analysis. Selection processes based on economic analyses have obvious advantages over other methods. Also, recourse to a computer is necessary for the analysis and ranking when more than a few projects and alternatives exist. A technique termed "dynamic programming" performs this task. The accuracy, however, depends on the accuracy of the benefit and cost values assigned to each element included in the analysis.

The Kentucky Department of Transportation first applied dynamic-programming techniques to the spot safety improvement program in 1974 (1). The application of dynamic-programming techniques to the resurfacing program was proposed as a way of optimizing expenditures. Since hundreds of candidate projects are recommended for resurfacing each year, it is difficult to select those that will yield the greatest benefit to the driving public. To apply dynamic programming or any other economic method to the resurfacing program, a reliable means of calculating benefits must be employed. This paper presents those procedures and criteria.

DYNAMIC-PROGRAMMING CONCEPT

The term "dynamic programming" was first used by Bellman to represent the mathematical theory of a multistage decision process (2). It is applied to allocate expenditures in a way that results in the maximum benefit. Three types of applications of dynamic programming are single-stage, multistage, and multistage that has a time factor. Single-stage programming is used to evaluate a single project that has several alternatives. Multistage programming involves selection of several projects that have several alternatives. Multistage dynamic programming that has a time factor is used when several projects and alternatives are considered and various time periods are involved. Multistage programming is currently being used in the safety improvement program in Kentucky. It was presumed to be also applicable to the resurfacing program.

Input to the model consists only of costs and benefits for a project and the useful life of the improvement. Costs are incurred by the highway agency, and benefits are gained by the road user (3). Costs associated with a project might include construction costs and annual maintenance costs. Benefits include savings of time and fuel, increased comfort (or ride quality), and accident reduction.

RESURFACING PROGRAM IN KENTUCKY

The Division of Maintenance is responsible for the statewide resurfacing program, which cost $12 million in 1977. The 12 highway districts select and rank resurfacing needs and submit a list of projects each year. A team composed of two engineers from the Division of Maintenance and one from the district reviews and evaluates the projects. The same two engineers from the Division of Maintenance evaluate sections throughout the state. According to a proposed form, maintenance sections are rated on a point system (maximum of 100 points) and are evaluated for service (15 points), condition (71 points), and safety (slipperiness) (14 points). A high point value indicates a need for resurfacing. Service evaluation is based on the annual average daily traffic (AADT) of the section. The maximum of 10 points is assigned to roads that have AADTs more than 10 501. An extra five points are added when traffic speeds are 22 m/s (50 mph) or more.

The subjective rating of pavement conditions (35 points) is based on raveling (spalling), cracking, patching, edge failure, base failure, out-of-section condition, and appearance. The proposed form would permit rating of severity as well as density (frequency) of the failure or deficiency. Rut depth from 9.5 to more than 22.2 mm (0.375-0.875 in) is assigned a maximum of 12 points. A roughness index (RI) is obtained by using the Kentucky method (4,5) or by correlation by using the Mays ride meter. Roughness ranges up to 24 points. If a roughness measurement cannot be obtained, ride quality is subjectively evaluated and rated as smooth (no points) to severely rough (22 points).

The safety rating is based on skid resistance. Pavements that have skid numbers (SNs) of 30 or less are assigned 14 points. The rating form used previously did not adequately weigh conditions that may warrant extreme measures when some important attribute was at an unacceptable level. The proposed form would require the addition of 100 points if the SN was 28 or less and the AADT was more than 1000. Similarly, 100 points would be added whenever the RI or rutting for a particular type of pavement and a given volume of traffic exceeded the values cited on the rating form. Resurfacing costs and district rankings are cited on each rating form.

PROCEDURE

Resurfacing costs and annual maintenance costs must be known, and benefits expected from accident reduction, improved comfort, and saving of time and fuel must be determined. Other inputs into the model include the probable life of the new surface, the interest rate, and unit costs of accidents, time, comfort, and fuel. These inputs can be easily changed from year to year as unit costs increase.

The effect of resurfacing on accident experience was found by analyzing the before-and-after accident data of approximately 3700 km (2300 miles) of road evaluated from 1973 through 1976. Correlations were also made between accident experience and pavement condition. This analysis was essential for projection of accident savings attributable to resurfacing.

An analysis was also made of the benefits to the road user from increased comfort. The cost of traveling over a newly resurfaced road was compared with that of traveling over a pavement in very poor condition. These costs were established from responses to questionnaires on which motorists indicated willingness to pay for travel on a new smooth pavement compared with travel on one in poor condition. The resulting costs per kilometer were converted to annual dollar benefits for highway sections, based on AADT and length.

Equations were also developed to compute benefits for time and fuel saving after resurfacing. Such information as pavement roughness, AADT, and vehicle speed were included in the analysis.

The resurfacing costs were those estimated by maintenance engineers for each section recommended for resurfacing. These costs were based on surface width, section length, type of surface, and many other factors. These costs represented present worth and were inputs into the dynamic-programming model.

A formula for annual maintenance costs was derived from annual maintenance costs for rural roads in Kentucky (6). Maintenance costs generally increase as a pavement ages. This was taken into account indirectly.

A present-worth factor was used to convert the annual maintenance cost and annual benefits to their present worth. For a given interest rate and number of years, a factor can be determined to convert a uniform series to its present worth (3).

Based on the costs and benefits computed for highway sections recommended for resurfacing in 1976, an appropriate computer program was prepared. An optimal priority listing of projects was derived. The projected benefits and costs of this optimal listing were compared with those of projects selected by using traditional methods.

SERVICE LIVES OF RESURFACING PROJECTS

Ideally, pavement overlays should be designed for a desired service life based on estimated traffic volumes. In this case, overlay types and thicknesses will vary by project and will influence resurfacing costs. The design period can be used as the estimated service life. To increase surface life, thicker, more durable surfaces should be used on roads that have heavy traffic volumes and heavy trucks. The overlay thicknesses for the resurfacing projects analyzed in this study were not based on structural designs but generally consisted of cost estimates for a standard 38.1-mm (1.5-in) surface course. The service lives of these overlays were estimated for various ranges of AADT. Service lives ranged from 7 years for AADTs of more than 8000 to 16 years for AADTs between 1001 and 4000. Lives of 12 years were estimated for sections that had AADTs of 400-1000 and 4001-8000. The actual designed service life can be used if known. The dynamic-programming model allows for input of the design life, which will then override the estimates above. In the past, standard 38.1-mm overlays have been customary. The program does allow for input of individual project design lives if this procedure is adopted in the future.

CALCULATION OF ROAD-USER SAVINGS

Before benefits can be computed for any highway improvements, some assumptions have to be made. If the condition of a pavement is known before it is resurfaced, the following questions must be answered before benefits can be computed:

1. How will the condition of the pavement change if no improvement is made to the pavement?
2. How will the condition of the pavement change if it is resurfaced?
3. What is the relationship between road-user costs and time as the overlay surface deteriorates over its useful life?
4. How can benefits be computed due to resurfacing for an overlay that has changing conditions throughout its life?

To answer these questions, two different types of assumptions were made to apply to the various types of road-user costs. The first is illustrated in Figure 1. Road-user costs are high at Cb after a pavement ages after time Ta. At time Tb, the pavement is resurfaced and the road-user costs immediately drop to level Ca. This reduction holds until time Tc, when road-user costs will increase either gradually or sharply. The second assumption applies to other types of road-user costs, which increase gradually after resurfacing until they reach a maximum level as shown in Figure 2. Point A represents the time shortly after a new pavement overlay. If no improvements are made to the surface, its condition will gradually worsen until it reaches point D. At this point, the pavement will not get much worse in terms of road-user costs; a road can only get so slick and rough and still be used. The road-user costs would then stay relatively constant at Cb until the road reaches point E in time. If the pavement is resurfaced at point B (road-user cost = Ca), the road-user costs would immediately drop to point G, which might be equated to no cost. The life of the new overlay will then be (Tb - Ta), or N. The road-user costs are then assumed to increase linearly over its life until they reach the peak value at point E. Another pavement overlay at point E would start the cycle once again.

If no improvements were made at point B, the road-user cost between times Ta and Tb could be represented by the area within the boundaries of BDEFG. This area gives the total road-user cost for time N. If the pavement is overlaid at time Ta, the saving in road-user costs is the shaded area represented by BDEG. By determining this area, the road-user saving or benefits can be found for the overlay life N.

The equation derived represents area BDEG. This area can be found by computing the area of the large rectangle (GHEF) and subtracting triangles 1 (GEF) and 2 (BHD). The final equation for BDEG total benefits (Bc) is as follows:

$$Bc = \left\{[(N)(Cb) - \tfrac{1}{2}(N)(Cb) - \tfrac{1}{2}(Cb - Ca)]\,[N - N(Ca/Cb)]\right\} Ff/N \qquad (1)$$

or

$$Bc = \left\{[(N)(Cb/2) - \tfrac{1}{2}(Cb - Ca)]\,[N - N(Ca/Cb)]\right\} Ff/N \qquad (2)$$

where Ff is a factor used to convert to present-worth benefits. The rest of the equation will give the average annual values of benefits for the project life, such as the following:

1. Average annual percentage of reduction in road-defect accidents due to resurfacing (accident benefit),
2. Average annual saving in comfort cost for the road user (cents per vehicle kilometer),
3. Average annual percentage of reduction in fuel cost, or
4. Average annual maintenance savings per vehicle kilometer.

Figure 1. First assumption of road-user costs versus time.

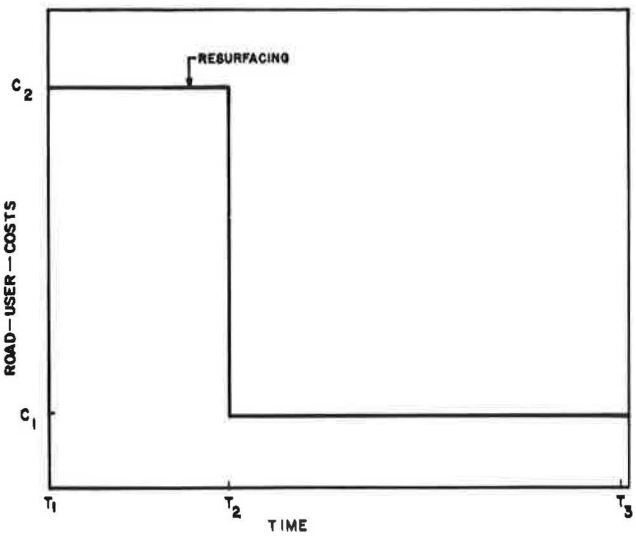
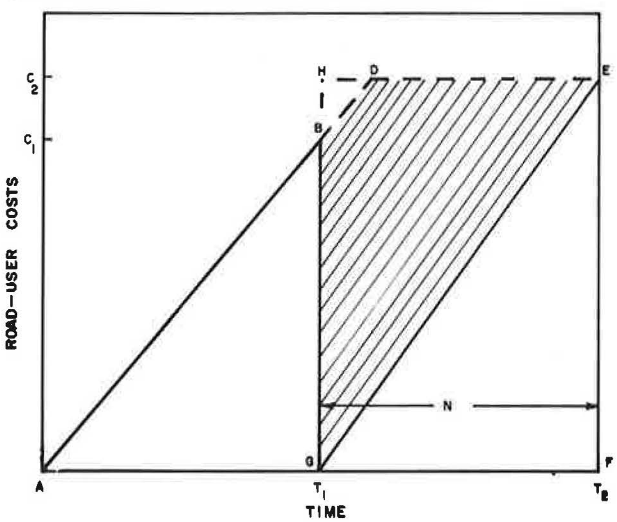


Figure 2. Second assumption of road-user costs versus time.



This assumption was used to estimate the present-worth benefits (road-user savings) in comfort costs, fuel costs, maintenance savings, and road-defect accidents. In all cases, road-user costs drop immediately after resurfacing. As time passes, the costs increase linearly until the maximum level is reached; then the road-user costs level off.

BENEFITS FROM RESURFACING

Increased Comfort

The value of comfort (or ride quality) to the road user has not been determined. In 1960, estimates of value for comfort were assumed by the American Association of State Highway Officials (AASHO) based on freedom of vehicle operation as follows (7):

1. Free operation, 0 cents/vehicle-km;
2. Normal operation, 0.3 cent/vehicle-km (0.5 cent/vehicle-mile); and
3. Restricted operation, 0.6 cent/vehicle-km (1 cent/vehicle-mile).

These unit costs are for operation of passenger cars in rural areas and for continuous movement on tangent or nearly tangent highways.

The benefit of any highway improvement that involves the comfort of a motorist may be approximated by observing the willingness of the motorist to pay for such benefits. One example of a superior highway facility may be Kentucky's toll roads (parkways). The average toll per kilometer (cars only) ranges from 0.9 to 1.5 cents (1.5-2.0 cents/vehicle-mile). The average cost for all toll roads is 1.2 cents/km (2.0 cents/vehicle-mile). The benefits to the motorist are greater on toll facilities when compared with the benefits from resurfacing other highway sections. A toll road offers not only a good riding pavement but also full access control, good alignment, improved safety, and reduced travel time. A reasonable benefit from a newly resurfaced road may be about half that of toll roads or around 0.6 cent/vehicle-km.

To gain a better understanding of the benefits derived from a newly resurfaced highway with respect to the improved comfort to the road user, a questionnaire was developed. The questionnaire asked what the motorist would be willing to pay to travel over a newly paved surface compared with a road in poor condition for a distance of 1.6-483 km (1-300 miles). The questionnaires were distributed to two groups. One group consisted of employees within the Kentucky Department of Transportation. There were 164 responses from this group. The other sample consisted of a selection from all licensed drivers. To obtain this sample, names and addresses of 1000 drivers were obtained from the driver's-license file. Letters not deliverable were sent to other drivers to assure a sample of 1000 drivers. Of the 1000 questionnaires sent, 203 were completed and returned. Although this is a response of only 20 percent, it was deemed an acceptable sample.

An average value per kilometer was calculated from each response. Responses from Kentucky Department of Transportation employees showed that the most common response (43 percent) was 0.6 cent/km. The median value and the mode were 0.6 cent/km. The average value was 0.8 cent/km (1.4 cents/mile). Results from the public at large were similar. Based on information available from other sources and the findings in this study, a benefit of 0.6 cent/km for increased comfort was chosen. This value corresponds to the benefit that would result from resurfacing a road in very poor condition.

The road-user cost of reduced comfort varies from 0 to 0.6 cent/vehicle-km, depending on the roughness of the pavement. The roughness may be expressed in terms of RI or present serviceability index (PSI). RI values normally range from about 300 for a smooth road to more than 1000 for a very rough road and correspond to a PSI from about 4.0 to about 1.5, respectively. The relationship between comfort costs and pavement roughness was assumed to be linear. As PSI decreases from 3.7 to 1.8, the comfort costs increase from 0 to 0.6 cent/vehicle-km. The comfort cost does not exceed 0.6 cent/vehicle-km. This value of the comfort cost in cents per vehicle kilometer before resurfacing corresponds to the value of Cb, which can be calculated as follows:

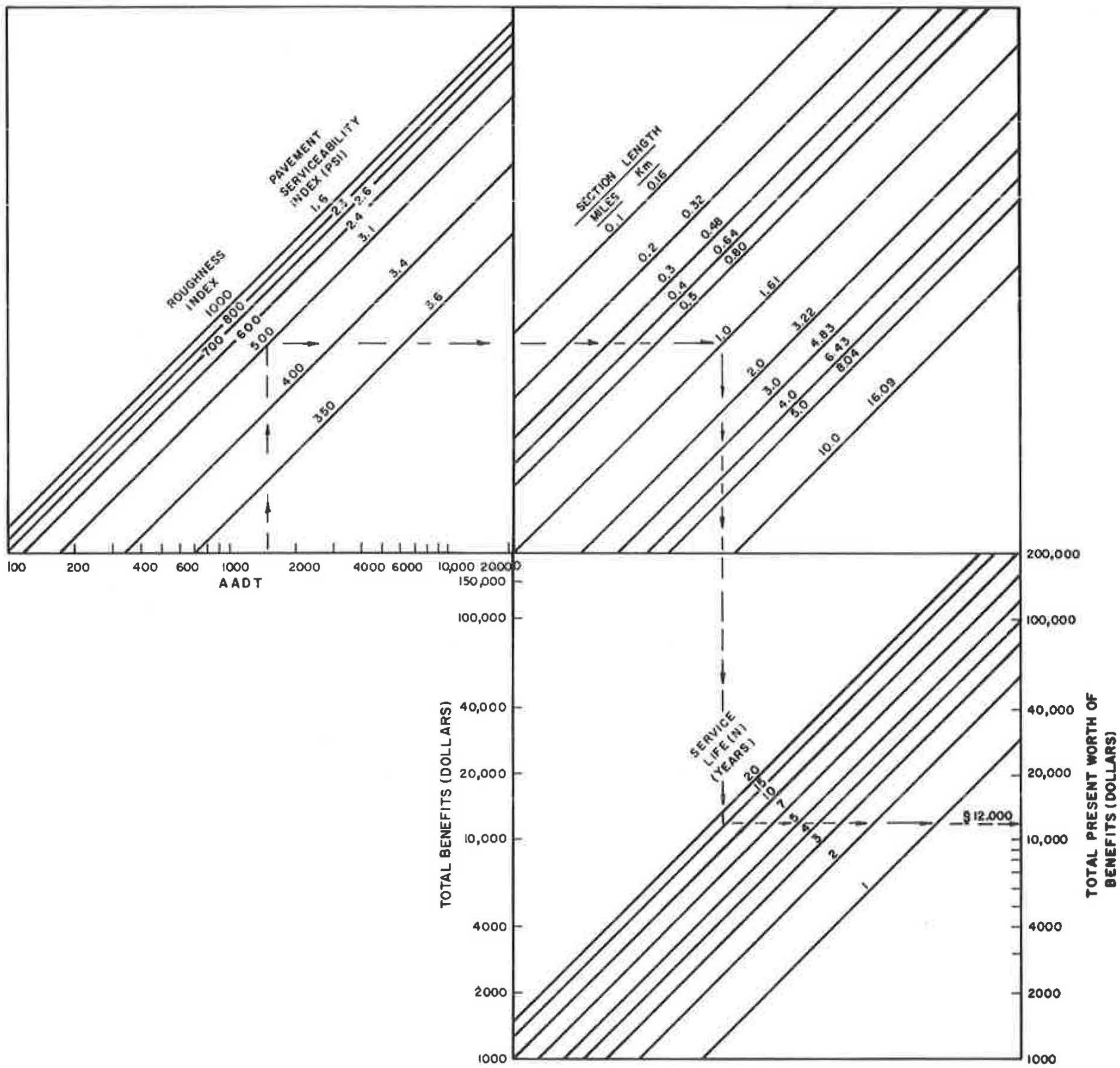
$$Cb = 0.0010(RI) - 0.31 \tag{3}$$

By using the procedure described previously for computing lifetime benefits of a pavement overlay, the formula for comfort benefits is the following:

$$Bc = [(NCm/2 - \tfrac{1}{2})(Cm - Cb)(N - NCb/Cm)]\,Fc/N \tag{4}$$

where Fc = (AADT)(365)(Ls)(PWF). Fc is a factor to convert to present-worth benefits. The rest of the

Figure 3. Nomograph for computing total comfort benefits due to resurfacing.



equation gives the average annual comfort cost (in dollars) per vehicle kilometer. The final equation then becomes the following:

$$Bc = [(NCm/2 - ½)(Cm - Cb)(N - NCb/Cm)(AADT)(365)(Ls)(PWF)]/N \quad (5)$$

where

Bc = present-worth benefit from driver comfort after resurfacing,
Cm = maximum possible comfort cost = $0.006,
Cb = comfort cost of pavement based on RI or PSI,
AADT = average annual daily traffic of the highway section,
Ls = section length (km),
PWF = present-worth factor, and
N = service life of the overlay (years).

To graphically determine the relationship among AADT, RI, section length, and comfort benefits, a nomograph was prepared (Figure 3). The nomograph gives approximate values, which will vary slightly from calculated values. To use the nomograph, enter the existing AADT on the highway section and draw a vertical line to the appropriate RI value. Proceed to the right to the section length and then down to the corresponding service life. Then read the total benefits at the right or left side of the page. (Similar nomographs were developed for the other savings, but they are not presented in this paper.)

Time Savings

Estimates of time savings by road users were determined on the basis of roughness of the pavement. Data used to develop this information were based partly on a 1972 report by McFarland in which vehicle speeds were associated with PSI (8). To further verify the effect of pavement roughness on vehicle speeds, vehicle speeds were observed before and after resurfacing a very rough section of road. Average speed after resurfacing was found to increase by about 4 m/s (8 mph). The pavement condition on the test section was assumed to be about as poor as will normally be encountered on a state-maintained road. The 4-m/s increase was used as the maximum when the expected speed increases after resurfacing roads that had an RI of more than 700 were estimated. The speed increase was related to RI and speed limit. No speed increases were assumed for a RI of less than 700. The maximum increase of 4 m/s occurs for speeds faster than 22.4 m/s (50 mph) and RIs of more than 950. Given the speed limit and RI, the computer program selects the approximate speed increase.

After the approximate speed increases expected after resurfacing a rough road had been determined, the formula for time savings for each vehicle was determined as follows:

$$St = Tb - Ta \tag{6}$$

where

- St = time savings (h),
- Tb = travel time before resurfacing (h), and
- Ta = travel time after resurfacing (h).

Travel times are calculated from the following equations:

$$Tb = L/Sb$$
$$Ta = L/Sa = L/(Sd + Sa) \tag{7}$$

where

- L = section length (km),
- Sb = vehicle speed before resurfacing (m/s),
- Sa = vehicle speed after resurfacing (m/s) (assumed to be the posted speed limit), and
- Sd = difference in speed due to resurfacing (m/s) (as determined by speed limit and RI).

The value of time was selected on the basis of a 1976 study by Agent (9). In that study, delay costs were found to be \$4.87/vehicle-h.

The annual time saving after resurfacing a rough highway was computed based on the section length, traffic volume, cost per vehicle hour, and time savings per vehicle. The formula for annual benefit due to time savings (B) is as follows:

$$B = (Tb - Ta\text{ hr})(\text{AADT vehicles/day})(365\text{ days/year}) \times (\$5.54/\text{vehicle-h}) \tag{8}$$

or

$$B = 1777.55(Tb - Ta)(AADT) \tag{9}$$

Vehicle speeds were assumed not to be affected on roads that had an RI of less than 700. Rizenbergs, Burchett, and Davis have shown that the RI on many roads remains less than 700 for the life of the pavement and that the average RI was 430 just after resurfacing and increased linearly to only 510 after nearly nine years in service (4). Although the RI of some roads may never exceed 700 due to timely resurfacing, other sections may be resurfaced only once every 20 years or longer. Roads that exhibited an RI of less than 700 before resurfacing will not show a time-saving benefit as calculated by the formula, since Tb would equal Ta. By using the present-worth factor (PWF), the present-worth benefit from time savings (Bt) was found to be as follows:

$$Bt = PWF(1777.55)(Tb - Ta)(AADT) \tag{10}$$

The present-worth benefit from time savings due to resurfacing can be quite significant. For illustration, a graphical procedure was developed to easily determine the approximate present-worth benefits of time savings that will result due to resurfacing. The vehicle speed after resurfacing is assumed to be equal to the speed limit. The difference in vehicle speeds is determined by the model as a function of speed limit and RI. Subtracting this value from the speed limit gives vehicle speed before resurfacing.

Fuel Savings

Resurfacing a pavement affects fuel consumption in two ways. Consider a pavement that is very rough and on which vehicles are forced to travel at a reduced speed: Resurfacing this pavement will result in an increase in vehicle speeds and a corresponding increase in gasoline consumption of as much as 13 percent (10). However, rough pavements cause vehicles to bounce, and it takes energy to induce vehicle motion. Therefore, more fuel is required to maintain speed on a rough pavement than on a smooth pavement. A rough pavement may require the driver to brake to avoid very rough spots. Thereafter, the driver must accelerate to the desired speed of travel. This added acceleration increases fuel consumption. Assuming a traffic mixture of 80 percent cars, 10 percent pickups or vans, and 10 percent large trucks (six tires or more), the adjustment for increased fuel consumption may be 36 percent at 20.1 m/s (45 mph) on a level road (10). The net effect of resurfacing may be a 23 percent reduction in fuel consumption after adjustment for extra fuel (13 percent) needed to maintain up to a 4.5-m/s (10-mph) higher speed on the road after resurfacing. This maximum of a 23 percent reduction in fuel use was used for resurfacing a pavement in very poor condition (rough).

The linear relationship between RI and reduction in fuel costs was developed based on an analysis of that information. The percentage of reduction in fuel use (Fl) can be computed by the following equation:

$$Fl = 0.0365(RI) - 11.52 \tag{11}$$

As RI increases from 317 to 950 (bituminous pavements), the percentage of reduction in fuel costs increases linearly from 0 to 23 due to resurfacing. By applying the equation for converting to present-worth benefits from fuel savings due to resurfacing, the equation is as follows:

$$Bf = [(FmN/2 - \tfrac{1}{2})(Fm - Fb)(N - NFb/Fm)]\, Ff/N \tag{12}$$

where

- Bf = present-worth benefits from fuel savings due to resurfacing a highway,
- Fm = maximum percentage of reduction in fuel costs (23 percent) due to resurfacing, and
- Fb = percentage of reduction in fuel costs based on RI before resurfacing.

Ff is a factor used to convert to present-worth dollars. The rest of the equation represents the average annual percentage of reduction in fuel savings due to resurfacing. The value of Ff must include the total traffic in vehicle kilometers that passes the section each year [(AADT)(Ls)(365)]. The fuel cost of these vehicle kilometers is found by assuming 65 cents/gal of gasoline and 5.1 km/L (12 miles/gal) for an average vehicle in Kentucky [national average of 5.0 km/L (11.85 miles/gal)]. The cost per gallon can be changed easily in the equation when it becomes out of date. The value of Ff is expressed as follows:

$$Ff = (\text{AADT vehicles/day})(365\text{ days/year})(Ls\text{ km})(1/5.1\text{ L/vehicle-km}) \times (0.17/L) \tag{13}$$

$$Ff = 12.17[(AADT)(Ls)\text{dollars/year}] \quad (14)$$

By using the base equation and the present-worth factor for any service life N, the final equation becomes as follows:

$$Bf = \{[(FmN/2 - ½)(Fm - Fb)(Fb/Fm)](PWF)(12.12)(AADT)(Ls)\}/N \quad (15)$$

where Bf is the present-worth benefit from fuel savings due to resurfacing a highway.

Annual Maintenance Savings

Comparisons of maintenance costs were made for highway sections before and after resurfacing. A relationship between pavement age and maintenance cost per lane kilometer per year for bituminous pavements in Kentucky was given in a 1974 research report (6). Annual costs per lane kilometer increased to about $311 during the 15th and 16th years and then diminished sharply. Obviously, resurfacing began to supplant regular maintenance at that time. Costs from that analysis were obtained from average costs per lane kilometer per year for 13 years; Interstates and toll roads were excluded. For this analysis, only ordinary maintenance costs were considered. (Physical improvements such as extensive overlaying are not considered ordinary maintenance.) Here annual costs were inflated to 1976 dollars by using the cost index for highway maintenance and operation as given by the Federal Highway Administration (11). The peak annual cost after 15 years was found to be $560/lane-km ($900/lane-mile) based on 1976 costs. This cost corresponds to a highway section in very poor physical condition that requires considerable maintenance each year.

The determining factors used for estimating maintenance costs were the subjective rating of pavement condition and rutting cited on the rating form. The point values given there were converted to a percentage of the maximum points possible (100 points).

By using the rating of deficiency points for pavements considered in the 1976 resurfacing program, all pavements were found to have ratings between 10 and 60. Maintenance costs range from 0 to $560/lane-km/year for deficiency ratings of 10-60. Based on this curve and Figure 2, the formula for present-worth benefits was determined as follows:

$$Bm = \{(MmN/2 - ½)(Mm - Ma)[N - N(Ma/Mm)]\}Fm/N \quad (16)$$

where

- Bm = present-worth benefits from maintenance savings due to resurfacing a highway section,
- Mm = maximum annual maintenance cost per kilometer before resurfacing ($560),
- Ma = annual maintenance cost per kilometer based on deficiency rating, and
- Fm = factor for converting to present-worth benefits [(PWF)(Ls)].

The value for annual maintenance cost per kilometer can be computed as follows:

$$Ma = 11.2(\text{deficiency rating}) - 112 \quad (17)$$

where the deficiency rating varies from 10 (new pavement) to 60 (pavement in very poor condition). Thus the final equation becomes the following:

$$Bm = \{(MmN/2 - ½)(Mm - Ma)[N - N(Ma/Mm)](PWF)(Ls)\}/N \quad (18)$$

Accident Savings

One of the benefits from resurfacing a pavement is the reduction in accidents. To determine the benefits in accident reduction, a relationship between accidents and pavement condition must be known. Comparisons were made between the accident data and pavement condition for highway sections evaluated from 1973 through 1976. This involved 513 sections that had a total length of about 3700 km (2300 miles).

Two types of accidents were found to be affected by resurfacing. The first relationship was between the condition of the pavement and the number of road-defect accidents. Pavements that had excessive cracking, base and edge failures, raveling, patching, out-of-section conditions, and rutting were found to have the greatest reduction in road-defect accidents after resurfacing. This reduction in accidents was then converted to an equivalent of 15 percent reduction in total accidents. The relationship was developed between percentage of reduction in total accidents (Al) and deficiency points (Dt) as follows: $Al = 18 - 0.3(Dt)$. Deficiency points range from 10 to 60 for accident reductions of 0-15 percent, respectively.

The reduction in road-defect accidents was expected to be the greatest after resurfacing and to gradually diminish over the life of the overlay. The following general equation was used for computing present-worth benefits:

$$Brd = \{[(NAm/2 - ½)(Am - Ap)(N - NAp/NAm)](Ca)(An)(PWF)\}/N \quad (19)$$

where

- Brd = present-worth benefits from reduction in road-defect accidents due to resurfacing,
- An = annual number of accidents on the section,
- Am = maximum percentage of reduction in accidents (15 percent),
- Ap = percentage of reduction corresponding to a particular deficiency rating, and
- Ca = cost of each accident ($4055).

The cost per accident was calculated by using the distribution of accident severities from police-reported accidents in Kentucky (1977). National Safety Council information on costs for each type of accident was applied to compute average cost per accident. Since virtually all proposed resurfacing sections are in rural areas (about 95 percent), only rural accidents were used to arrive at the costs of a representative accident. The average cost per accident was computed to be $4055.

Whereas resurfacing will cause a reduction in road-defect accidents, improved skid resistance of pavements will also reduce wet-pavement accidents. A relationship between accidents and pavement friction has been reported by Rizenbergs, Burchett, and Warren (12). The percentage of wet-weather accidents was found to be greatest on pavements that had low skid resistance. Percentages of wet-pavement accidents decreased as the SN increased to about 40. If a pavement had an SN less than 40 before resurfacing, the improved skid resistance after resurfacing would result in a reduction in wet-pavement accidents. The results of that study were used to compute the relationship between percentage of reduction in total accidents (Ar) and SN as follows: $Ar = 40 - SN$. In the range of SNs between 20 and 40, the reduction in wet-pavement accidents was about 50 percent, which corresponds to about 20 percent reduction in total accidents (12).

Class 1, type A bituminous concrete is the predominant mixture used in resurfacing, and the performance of this type of surface was used to determine when the skid resistance of an average pavement may reach an SN of 40 (after 3.7 million vehicle passes). The number of years wet-pavement accidents may remain reduced for various AADTs was found to be about five years for AADT of 400 or less, about seven years for AADT between 4001 and 8000, and three years for AADT more than 8000. A maximum of five years was selected in determining total accident reductions.

The general equation used for computing present-worth benefits was as follows:

$$Bww = (Ar)(An)(Ca)(PWF) \quad (20)$$

where Bww is the present-worth benefits from reduction in wet-pavement accidents due to resurfacing and Ar is the percentage of reduction corresponding to a particular SN.

The accidents that may be reduced due to resurfacing consist primarily of road-defect and wet-pavement accidents. The procedure given here involves separate calculation of each component of accident benefits. After both benefit values are found, they are to be added to yield total present-worth accident savings.

Other Benefits

In addition to benefits from accident reduction, improved comfort, time savings, and fuel savings, there are other benefits associated with resurfacing of a highway. Examples of other such benefits include savings in vehicle maintenance costs, reduction in highway noise, and reductions in vehicle-related air pollution. These benefits are very difficult to quantify in terms of monetary benefits and thus were not included in the dynamic-programming model.

RESURFACING COSTS

Resurfacing costs are estimated annually for each road section recommended for resurfacing by the highway districts. The estimates are based on section length, highway width, number of lanes, type of proposed surface, and the availability and cost of materials and labor. In the 1976 resurfacing program, 1670 km (1037 miles) of road were considered; the total estimate was $29 615 000. The average statewide cost of resurfacing based on those estimates is $8825/lane-km ($14 200/lane-mile). This corresponds to an average cost of $17 600/km ($28 400/mile) for two lanes. The resurfacing costs used in the dynamic-programming model were the estimates given for each project.

DYNAMIC PROGRAMMING

Input

Input into the dynamic-programming model consists mostly of information and data available from the pavement-rating forms and includes location (district, county, route, and milepost), deficiency rating, RI, SN, AADT, speed limit, section length, and resurfacing cost. The total number of accidents during the previous year is an added input.

Other information needed for the program includes interest rate of money (assumed to be 8 percent in this study), average cost per accident ($4055 for rural roads in Kentucky for 1977), and number of locations being considered. Because the budget for resurfacing in each district is arrived at essentially on the basis of a formula described earlier, dynamic programming was applied to highway sections recommended for resurfacing by each district and the district's budget.

Output

A listing of benefits and costs and the benefit/cost ratios for each highway section are in the first part of the program output. A statewide listing of highway sections ordered by benefit/cost ratio is also contained in the program output. All benefits and costs and cumulative benefits and costs are cited there. This listing could be used to determine project priorities based entirely on the benefit/cost ratios. The final section of the program output contains listings of projects selected for each district based on allotment of funds for resurfacing in that district. The total cost and benefits and the benefit/cost ratios for the selected projects are also cited. All projects considered are listed, but only the costs and benefits of projects selected for resurfacing are shown.

PRESENT PROCEDURES COMPARED WITH DYNAMIC PROGRAMMING

A computer printout was also obtained that lists all 233 projects according to the benefit/cost ratio. The highest ratio was 20.10 and the lowest was 0.18. Information includes the location identification number (1-233), section length, project benefits, project cost, cumulative benefits, cumulative cost, cumulative benefit/cost ratio, and cumulative length. There were 251 km (156 miles) of road with benefit/cost ratios in excess of 4.0, and 1249 km (776 miles) of the 1520 km (944.9 miles) of road being considered had benefit/cost ratios more than 1.0. Cumulative costs for the 233 projects were $22.5 million, and cumulative benefits were more than $58 million. This corresponds to an overall benefit/cost ratio of 2.58.

The various benefits (savings) associated with resurfacing all projects were also detailed. When the projects were combined, 42 percent of the benefit ($24.5 million) resulted from fuel savings and 34 percent ($19.7 million) from comfort benefits. Other benefits include 15 percent ($8.6 million) for time savings, 6 percent ($3.3 million) for accident reduction, and 4 percent ($2.1 million) for maintenance savings. Of the 233 projects, only 42 had benefits from time savings (pavements with an RI of more than 700). All projects showed benefits due to improved comfort and maintenance savings; 53 sections showed no benefits from accident savings.

The results of selecting projects by dynamic programming for each district were compared with the results from procedures now used by the districts and the Division of Maintenance. The current procedure of selecting projects yielded total benefits that amounted to about $27.7 million compared with benefits of $36.1 million derived from projects selected by dynamic programming. The cost of the projects selected by dynamic programming was also slightly lower ($8.5 million compared with $8.6 million).

The benefit/cost ratio of projects selected for resurfacing in 1976 was 3.21 compared with 4.22 if the selection of projects had been made by dynamic programming on the basis of budget allocation to each district. Dynamic programming, therefore, would have yielded a 30.4 percent increase in benefits and would have reduced costs by 0.9 percent. The overall improvement in the benefit/cost ratio would have been 31.5 percent if dynamic programming had been applied.

Projects Selected on a Statewide Basis

If projects had been selected by benefit/cost ratio alone on a statewide basis by using funds allocated to the resurfacing program in 1976 ($8.6 million), the projects selected would have had an overall benefit/cost ratio of 4.52. This is somewhat higher than the ratio of 4.22 obtained by using dynamic programming based on budget allocations by district and is substantially higher than the 3.21 realized in 1976 by selecting projects according to established procedures. If the statewide budget of $8.6 million had been spent strictly according to the priority ranking based on the total deficiency ranking, the resultant benefit/cost ratio for all the projects would have been 3.29.

Comparison of Dynamic Programming with Benefit/Cost Method

Tests were made to compare the choice of projects selected for resurfacing by dynamic programming and by their benefit/cost ratios alone. A comparison by using one budget for the entire state ($8.6 million) was used. As stated earlier, an overall benefit/cost ratio of 4.52 was obtained by using a benefit/cost procedure (selection of projects based entirely on benefit/cost ratios). The results by using dynamic programming depended on the increment size used in the program. The amount of computer storage available becomes a problem if a small increment size is used. However, if the increment size is larger than some of the project costs, the efficiency of the program is decreased. Increment sizes of $50 000, $25 000, and $10 000 were used. This compares with an increment size of $1000, which was used for each individual district budget. For the $50 000 increment, a benefit/cost ratio of 4.43 was obtained. The benefit/cost ratio increased to 4.50 for the $25 000 increment size and 4.51 for the $10 000 increment size. This analysis showed that dynamic programming also yielded identical results compared with the benefit/cost method when an appropriate increment size was used.

SUMMARY AND CONCLUSIONS

The objective of this study was to develop an economic analysis and a dynamic-programming procedure that would assist in optimizing expenditures in the pavement-resurfacing program in Kentucky. Procedures were developed to compute benefits and costs of proposed projects and to determine which highway sections should be resurfaced under a given budget. A computer program was written to select an optimal list of projects for resurfacing based on road-user savings in accidents, travel time, comfort, maintenance costs, and fuel. Costs included in the model were resurfacing costs. Projects selected by the districts and projects selected by the Division of Maintenance for resurfacing in 1976 were evaluated by using the dynamic-programming model. An additional benefit of more than $8.4 million would have resulted from the use of the dynamic programming developed in this study. The benefit/cost ratio of sections selected for resurfacing by the current procedures was 3.21 compared with that of 4.22 if dynamic programming had been used. Projects selected by the Division of Maintenance had a much higher benefit/cost ratio (4.37) compared with projects selected by the districts (2.38). Projects selected on a statewide basis by dynamic programming or their benefit/cost ratio in 1976 would have resulted in a higher benefit/cost ratio (4.52) as compared with selections based on budget allocations to the districts (4.22). Selection of projects on a statewide basis and by using the total deficiency rating of pavements would have yielded a lower benefit/cost ratio (3.29). The economic analysis showed a very similar choice of projects when dynamic programming was used compared with selecting projects based solely on their benefit/cost ratio. The cost data included in this study should be updated before the program is used. The program is written so that the cost data can be easily changed.

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REFERENCES

1. J.G. Pigman, K.R. Agent, J.G. Mayes, and C.V. Zegeer. Optimal Highway Safety Improvement Investments by Dynamic Programming. Division of Research, Kentucky Bureau of Highways, Frankfort, Aug. 1974.
2. R. Bellman. Dynamic Programming. Princeton Univ. Press, Princeton, NJ, 1957.
3. E.L. Grant and W.G. Iresan. Principles of Engineering Economy. 5th ed. Ronald Press, New York, 1970.
4. R.L. Rizenbergs, J.L. Burchett, and L.E. Davis. Pavement Roughness: Measurement and Evaluation. TRB, Transportation Research Record 471, 1973, pp. 46-61.
5. J.L. Burchett, R.L. Rizenbergs, and T.A. Moore. Surface Dynamics Profilometer and Quarter-Car Simulator: Description, Evaluation, and Adaption. Division of Research, Kentucky Bureau of Highways, Frankfort, Jan. 1977.
6. D.D. Newberry, J.G. Mayes, and J.H. Havens. Highway Pavement Maintenance Costs and Pavement Type Selection. Division of Research, Kentucky Bureau of Highways, Frankfort, Dec. 1974.
7. Road User Benefit Analyses for Highway Improvements. AASHO, Washington, DC, 1960.
8. W.F. McFarland. Benefit Analysis for Pavement Design Systems. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 123-13, 1972.
9. K.R. Agent. Development of Warrants for Left-Turn Phasing. Division of Research, Kentucky Bureau of Highways, Frankfort, Aug. 1976.
10. P.J. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971.
11. Highway Statistics. FHWA, U.S. Department of Transportation, 1974.
12. R.L. Rizenbergs, J.L. Burchett, and L.A. Warren. Relation of Accidents and Pavement Friction on Rural, Two-Lane Roads. TRB, Transportation Research Record 633, 1977, pp. 21-27.

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Implementation of an Urban Pavement Management System

P.C. CURTAYNE AND T. SCULLION

The city of Johannesburg is nearing the end of the first stage in the implementation of a pavement management system. This work was centered on assisting the road authority in identifying roads that require resealing or overlays. It is based on a model that relates pavement condition to maintenance requirements and to the timing of future inspections. This system has already been used to good effect in preparing the maintenance program for 1981. This paper describes the nature and scope of the work in order to show what techniques have been found useful and what problems can be expected. An important feature of this system is its basic simplicity. It requires only the minimum amount of information, which is reasonably easy to collect. The favorable reaction by the staff to this innovation is discussed and special attention is focused on the threat that the introduction of more-mechanized methods of pavement assessment poses to the job satisfaction of road inspectors.

Over the past decade, considerable attention has been given to the development of pavement management systems. Yet the full-scale implementation of such systems is still in its infancy. In fact, in 1979 Finn (1) reported that no agency in the United States had yet implemented a pavement management system on a complete roadway network.

In South Africa, there is growing interest shown by both rural and urban authorities in developing a more formal approach to the management of their pavement networks. However, the Johannesburg City Engineer's Department is the only urban authority that has made significant progress in implementing a suitable system.

This paper describes the nature and scope of this work and emphasizes aspects that are thought to be unique to this system. Some of the problems encountered during implementation are highlighted, and an attempt has been made to assess the advantages of the system from the point of view of the maintenance engineer and to indicate what the future prospects are.

BACKGROUND AND OVERVIEW OF EARLIER WORK

The municipality of Johannesburg has an extensive pavement network made up of some 2700 km. These are divided into 4500 streets that total 22 000 city blocks. Currently, special maintenance treatments (i.e., overlays and surface treatments) are applied to approximately 2.0-2.5 million m^2 of pavement (about 9 percent of the total) each year.

The standard procedure for the selection and scheduling of these projects for special maintenance would take the following course:

1. An annual inspection of the pavements was made by the road inspector. Since it was impossible to inspect all the pavements in the network each year, a selection was made based on the road inspector's knowledge of probable problem areas and sometimes with the advice of or requisitions by colleagues such as the district engineers.
2. From the results of this inspection, a preliminary list of projects was prepared for the maintenance engineer.
3. The maintenance engineer then prepared the final program of work for the following year on the basis of this list, available resources (particularly the budget), and auxiliary discussions and inspections of doubtful cases.

Records were kept of special maintenance work carried out on the various streets by way of a card-index system.

The inadequacies of these procedures were well recognized during the 1970s. The card-index system was satisfactory for answering such queries as when South Street was last resurfaced but it was unable to cope with such queries as which streets had single seals older than six years. The procedure for identifying maintenance requirements had many drawbacks, which are discussed in detail by Gordon and Curtayne (2). In summary, they are the lack of consistency in the rating of pavements, the lack of control over the choice of roads to be inspected, and the lack of records of the results of past inspections.

The first innovation to be introduced was the establishment of a computerized street inventory (in association with other sections of the department that had similar problems) and the addition of the history of special maintenance from the manual card-index file.

The second development was the creation of a pavement management system that could operate in conjunction with this inventory. Initially, a model had to be established by which the maintenance requirements could be assigned from a description of the condition of the pavement. This model is to a large extent a formal expression of local experience and the policy of the department and takes the form of sets of rules similar to the decision trees described by Finn (1).

The method of establishing the model involved the assessment of the maintenance requirements of a set of pavements by a panel of raters (made up of experienced engineers and road inspectors) and the correlation of these assessments with quantitative descriptions of the pavement distress. [Full details of this procedure have been given by Gordon (3). Summarized versions have also been published in conference proceedings (2,4).]

The subjective manner in which this model was developed was necessary for the following reasons:

1. It was thought that the most meaningful information could be derived from local experience. There are few objectively determined relationships of this type given in the literature, especially ones that would be suited to local conditions.
2. To be accepted by the officials of the department, the model had as much as possible to be compatible with their current practices and policies.
3. The demands of the model (i.e., input and output and computer requirements) had to be compatible with the resources and organization of the department.

The model was completed and tested in 1978. The next step was to implement it in the working environment of the department. Because of staff changes that affected key personnel, this was only done toward the end of 1979. Full details of the implementation to date are given elsewhere (5) but are summarized in the next section.

Note that the implementation of this system was

Figure 1. Computer output of assessments.

```
                        RECOMMENDED   MAINTENANCE   REQUIREMENTS
                        ***************************************
     TOWNSHIP- TURFFONTEIN
     *********
STREET- GARDEN STREET             1  2  3  4  5  6  7  8  9    SQ M      RAND     TYPE   YEAR
  GERMAN ROA TO XAVIER ROA        SS                           5200      6240     SS     1980
  XAVIER ROA TO CORONATION           SS                        3200      3840     SS     1980
  CORONATION TO EASTWOLD #           SS                       15700     18840     SS     1980
  EASTWOLD # TO SIDE ROAD         SS                           3200      3840     SS     1980
                                                              -----     -----
                                                              27300     32760
                                                              -----     -----
STREET- BELLAVISTA ROAD
  TENNYSON D TO SOUTH RAND                 SL                  4800      2400     SS     1980 ***
  SOUTH RAND TO VAN HULSTE                 SL                 10600      5300     SS     1980 ***
                                                              -----      ----
                                                              15400      7700
                                                              -----      ----
STREET- DE VILLIERS STREET
  DONNELLY S TO LEONARD ST                          VR         8000     17600     VR     1983
  LEONARD ST TO CORNWALL R                                VR  14200     31240     VR     1983
                                                              -----     -----
                                                              22200     48840
                                                              -----     -----
                                                              -----     -----
STREET- ALLIN STREET
  BELLAVISTA TO SIDE ROAD                                 SL   4800      2400     RI     1982 ***
                                                               ----      ----
                                                               4800      2400
                                                               ----      ----
```

aimed at fulfilling the immediate needs of the department as soon as possible. Therefore the main requirement has been one of simplicity. Accordingly, no instruments are used in field rating and no mathematical techniques [such as those described by Karan and Haas (6)] are used in assessing priorities. However, the nature of the system is such that more-sophisticated techniques can be incorporated in the future.

IMPLEMENTATION

Output Specifications

Of the various goals associated with pavement management systems, it was decided that the most important for the first stage of implementation was to assist the road inspector in planning the inspection of the pavements and in selecting those to be recommended for special maintenance. Although the system was planned to incorporate further applications in the future (such as those discussed below), the emphasis was on introducing a useful application as soon as possible. In this regard, three computer output forms were designed.

Assessment of Maintenance Requirements

During inspection, the road inspector describes the condition of the pavement according to a set format (i.e., condition rating) and also adds his or her own assessment of the maintenance requirements. The model, by using this information together with data on the inventory (e.g., traffic and road widths), produces assessments of the maintenance requirements, which are displayed in the form of the output shown in Figure 1. For each township, the streets that have been rated are listed. These streets are broken down into lengths that have the same rating (demarcated by intersections with cross roads). The assessment is given in terms of the following:

1. Type of treatment (e.g., slurry, single seal, double seal, overlay), denoted by a symbol (e.g., SL, SS, DS, VR);
2. Urgency or priority of treatment (1, highest priority; 9, lowest; 10, pavement does not require maintenance in the following year but should be reinspected at a later date, which is stored in the inventory for future use);
3. Amount of work involved in terms of area and cost;
4. Opinion of the inspector, which gives the recommended type of treatment and year in which it should be done; and
5. Whether opinion of the inspector differs substantially from the result obtained from the model, in which case asterisks are printed in the last three columns. This acts as a warning that the input data, the model, or the inspector's assessment may be in error. On the other hand, the inspector may have taken factors into account that are outside the scope of the model. The inspector's opinion would therefore prevail. This is regarded as an important feature of the pavement management system and will be discussed again below.

The layout of this form provides a visually acceptable presentation. It is easy to see which streets have urgent maintenance requirements and how different lengths of pavement within one street differ with respect to these requirements.

Summary of Maintenance Requirements

The summary of maintenance requirements gives the total amounts for the various types of maintenance treatment for each priority level (Figure 2). Separate amounts are allocated for each maintenance type in the budget. This output can then be used to determine to which level of priority work can be undertaken for each type of treatment.

Recommended Inspection Schedule

As stated above, one of the main aims of the recom-

Figure 2. Summary output of maintenance requirements.

SUMMARY OF RECOMMENDED MAINTENANCE EXPENDITURE

PRIORITY		SLURRY SEAL	SINGLE SEAL	DOUBLE SEAL	VENEER CARPET	SLURRY + S.S.	+ D.S.
	1	26000	60000	25000	12000	500	2000
		(13000)	(66000)	(37500)	(24000)	(800)	(3800)
	2	16000	30000	20000	48000	000	000
		(8000)	(33000)	(30000)	(36000)	(000)	(000)
	3	10200	2500	14000	5320	1000	4000
		(5100)	(2750)	(21000)	(10640)	(1600)	(7600)
	4	20930	36000	8600	2765	1550	10250
		(10465)	(39600)	(12900)	(5530)	(2480)	(13475)
	5	14872	9656	2300	7632	180	600
		(7436)	(10621)	(3450)	(15264)	(288)	(1140)
	6	14276	82597	75489	13120	2530	000
		(7138)	(90965)	(113233)	(26240)	(4048)	(000)
	7	105326	43290	18290	5392	000	000
		(52663)	(47619)	(27435)	(12784)	(000)	(000)
	8	17932	81070	5489	38790	000	8290
		(8966)	(89177)	(8233)	(77580)	(000)	(15751)
	9	62970	21000	3600	29576	2590	12056
		(31485)	(23100)	(5400)	(59152)	(4144)	(22906)
TOTALS							
AREA		288506	366213	172768	163595	8350	37196
RAND		144253	402834	259152	327190	13360	70672

* AREA *
* (RANDS) *

Figure 3. Output of inspection schedule.

```
RECOMMENDED INSPECTION SCHEDULE
*********************************
TOWNSHIP- ROBERTSHAM
********
STREET- MOUNT IDA ROAD                       REASON
  LANDSBOROU TO TOBY STREE                   SLURRY =4YRS
  JERMYN STR TO LANDSBOROU                   SLURRY =4YRS
STREET- XAVIER STREET
  SIDE ROAD  TO SWORDER ST                   REINSPECTION
  SWORDER ST TO BITCON ROA                   REINSPECTION
STREET- COLLEGE ROAD
  FIRST AVEN TO SWIFT ROAD                   VENEER >10YRS
  SWALLOW RD TO FIRST AVEN                   VENEER >10YRS
STREET- PAUL KRUGER STREET
  CHURCH STR TO SCHOEMAN S                   VENEER =10YRS
  SCHOEMAN S TO PRETORIUS                    VENEER =10YRS
  PRETORIUS  TO BOOM STREE                   VENEER =10YRS
```

mended inspection schedule is to control and assist the inspection of pavements. The output shown in Figure 3 lists streets that should be inspected during the following cycle, which is based on two criteria:

1. Age of the surfacing: Inspections are recommended for slurry seals older than three years, all seals older than six years, and overlays older than nine years.
2. Relation to previous inspections: A priority of 10 assigned by the model means that maintenance is not required and a date for the next inspection is recommended. This date overrides the age requirements; i.e., it further reduces the number of pavements to be inspected.

Also included in this output is the reason why the road is due for inspection. [Note that these output forms are in a state of adaptation. Several versions have been tried in order to find the format that best suits the conditions in practice.]

Implementation Activities

Apart from the preparation of these computer programs, there were various other activities that required special attention. Some aspects of these were the following:

1. Training of inspectors: Special training is required for the preparation of the input form and for the use of the output forms. The input form requires an accurate understanding of the terms used. Although most of these are standard (7), their special application for this work was set out in a guide (8) for ready use by field inspectors. The involvement of the road inspector in preparing both the input form and the guide was an important part of this phase of the work.

2. Checking and completing the inventory: When the inventory came to be used, it was discovered that it contained many omissions and errors. A laborious process of checking and correcting the inventory against a street map was necessary. The inventory was completed on a suburb-by-suburb basis. Initially a set of suburbs in a region was completed in order to evaluate the system, to train officers in its use, and to detect any problems in operation.

Current Status

At present about 90 percent of the inventory has been completed, half of which has been checked and corrected. This inventory has already been used to identify pavements with surfacings older than the prescribed limits. These were duly inspected and evaluated, both subjectively and by the model. The exercise produced sufficient information to make a substantial contribution to the maintenance program for 1981 (by identifying some 1.8 million m^2 of the 2.5 million m^2 of work to be done). It also provided an important opportunity to reevaluate the model. There were numerous discrepancies between the assessments by the model and the opinion of the inspector. These were found to have three sources:

1. Misunderstanding of the procedure for field rating, which was greatly improved by subsequent training by using the new guide (8);
2. Normal differences of opinion, which, as shown by Gordon (3), can be very wide, even among experienced engineers--one of the aims of the pavement management system is to deal with these differences; and
3. Inadequacies of the model, indicated by some of the errors.

The assessments supplemented next year include

1. Those identified when more pavements reach the age limits (since the inventory will then be complete, this will include all such pavements);
2. Those with reassessments based on the analysis of this year's assessments; and
3. Those with any additional assessments that can be accommodated by the inspector.

In this way a complete cover of at least one assessment should be achieved within a few years while use is still made of the system in the interim period. After this has been achieved, the system can be used to its best advantage, which will give a complete picture of the inspection and maintenance requirements.

Future Development

A full pavement management system would encompass many features such as accounting and design. However, two applications envisaged and pertinent to problems currently being experienced by the department are the following:

1. Planning of resources: Separate teams are responsible for the various maintenance activities, such as slurry sealing and overlaying. Such teams take time to assemble and disband. Advance planning is necessary to provide an even flow of work through the years and timely enlargement or reduction in the size of the team.
2. Provision of pavement statistics: An area in which such statistics may be needed is the motivation for new equipment. For example, if a planer is being considered, it is important to know the area of asphalt surfacing, particularly in the central business district.

Apart from the conventionally stated benefits of pavement management systems, the above-mentioned examples underline the value of formal procedures for storing and processing pavement data.

EVALUATION

General

Despite several problems, a satisfactory level of implementation was achieved fairly quickly. This success is attributed in part to the simplicity of the first phase. The inventory contains only the minimum amount of information, which was reasonably easy to collect. A second reason for the success is that the system could be used in the practical decision-making process of the department during the early stages of implementation, even though compromises had to be made. Of particular importance was the fact that this was apparent to all levels of management.

Implementation Problems

The problems encountered centered on the fact that innovative activities such as the introduction of a management system tend to be outside the traditional duties and organizational structure of road authorities. Specific problems were as follows:

1. Staff changes: A large part of implementation concerns the motivation and training of staff who will have to carry out the work in the department. These are required at all levels of management (i.e., senior, middle, and field management). A change of staff at any of the levels can result in a lapse of motivation from that quarter. This can impede the momentum of the work severely and jeopardize the project. In the implementation of this system, changes have occurred at all three levels. However, apart from temporary setbacks, continuity has been maintained.
2. Organization: An innovation such as this requires the participation of staff and communication between them outside the normal organization channels, which makes it difficult to control and coordinate the various aspects of the work. Although this caused problems from time to time, they were largely overcome by individual motivation (from a perception of the value of the work) and from encouragement by senior management.

In general it was found that although goal setting and organizational considerations are of significance, the reciprocal criteria of motivation and momentum are crucially important. Motivation was promoted by (a) education at all levels regarding the nature and purpose of the work, (b) participation at all levels in goal setting and formulating of future work, and (c) early results that were meaningful to those concerned. Sustained momentum generates motivation. If this momentum is sufficiently great, the difficulty of bridging problems such as staff changes or organization conflicts is much reduced.

Reaction of the Department

Members of the department feel that developments to date have already brought about a marked improvement in the previous situation, in which pavements were often selected for maintenance on an arbitrary basis. They also believe that once this phase is finalized, far better control will be possible and maintenance programs will be set up with increasing confidence; improved resource planning will also be possible.

Their chief reservation is that the task of the road inspector now becomes much more mechanical and tedious. Although the number of roads to rate each year will decrease, their assessment will take much longer. Also, the road inspector has, until now, been the main person to formulate the annual program and has therefore had a job of high responsibility and interest. It is feared that if this were taken away, the road inspector would lose much of his or her interest and the quality of the assessments would consequently suffer.

Future Role of Road Inspector

There are two main responses to fears that the job quality of the road inspector would suffer as the result of the improved pavement management system:

1. Because the assessments have been formalized to a much higher degree than before and because it will be known beforehand which roads need to be inspected, the task of inspection can be decentralized much more readily and can be shared by the four districts. If 25 percent of the network needs to be inspected annually, each district would be allocated about 150 km of road, which is not too onerous. The role of the road inspector would then be one of controlling the quality of work (probably by spot checks).
2. Within the new system, the road inspector is still the primary person responsible for the condition of the pavements in the network; it remains the inspector's responsibility to recommend the maintenance program each year. The outputs from the model are aimed to help in this task, but when differences occur between the model's output and the inspector's own assessment, the inspector has the opportunity to make the overriding decision. The advantage is that, because of the greater formalization, the inspector's work is much more easily supervised by the maintenance engineer. For example, the road inspector must be able to explain the decisions made.

Because of retirement, there has been a recent change of staff in the department, and a younger officer who has a less-thorough knowledge of the road network than his predecessor has been appointed the road inspector. This situation has been cited as one of the important reasons for needing a more formal pavement management system (2). Even at this stage of development, the system proved more effective in accommodating a change such as this.

CONCLUSIONS

Substantial progress has been made in the implementation of a pavement management system for the city of Johannesburg. This system has already been used to assist in preparing the program of special maintenance work for the following year. Much of this progress is attributed to the basic simplicity of the system and its early use within the department. Several problems were encountered but have to a large extent been resolved. However, they did emphasize the importance of (a) the initial use of simple applications that will have practical use early during development, (b) motivation of all levels of staff, and (c) sustained momentum of implementation.

The reaction by the department has been positive. The potential of even the first phase of development is seen as a great improvement on the somewhat arbitrary nature of previous methods.

Care should be taken to ensure that this system improves rather then detracts from the job value of the road inspector. This is achieved by ensuring that the previous responsibilities with the system are maintained, which will facilitate rather than dictate the inspector's decisions. This not only obviates negative consequences such as the deterioration of the quality of the field data but also has important positive effects, namely, that the experience and initiative of the officer can be fully exploited while he or she nevertheless is still under the explicit control of higher management.

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REFERENCES

1. F.N. Finn. Management System for Pavement Maintenance. Transportation Engineering Journal of ASCE, Vol. 105, No. TE4, July 1979, pp. 457-472.
2. R.G. Gordon and P.C. Curtayne. Towards Rational Determination of Maintenance Needs and Priorities for Pavements. Proc., 9th Conference of Australian Road Research Board, Brisbane, Australia, Vol. 9, Part 4, 1978, pp. 101-106.
3. R.G. Gordon. Systematic Determination of Special Maintenance Requirements for Minor Urban Roads in Johannesburg. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa, Tech. Rept. RC/1/78, 1978.
4. R.G. Gordon and G.H.W. Papendorf. Determination of Special Maintenance Requirements for Urban Pavements. Proc., 3rd Conference on Asphalt Pavements in South Africa, Durban, 1979, pp. 9-18.
5. T. Scullion and P.C. Curtayne. Implementation of a Pavement Management System for the City of Johannesburg. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa, Tech. Rept. RC/3/80, 1980.
6. M.A. Karan and R. Haas. Priority Programming for Municipal Pavement Management. Australian Road Research, Vol. 9, No. 1, March 1979, pp. 38-50.
7. Standard Nomenclature and Methods for Describing the Condition of Asphalt Pavements. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa, TRH 6, 1980.

8. T. Scullion. Guide to Describing Pavement Condition of Minor Urban Roads in Johannesburg. National Institute for Transport and Road Research, Council for Scientific and Industrial Research, Pretoria, South Africa, Tech. Rept. A/PAD/03.1, 1979.

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Pavement Performance Modeling for Pavement Management

R. DARYL PEDIGO, W. RONALD HUDSON, AND FREDDY L. ROBERTS

Systematic pavement management requires estimates or predictions of future pavement performance so that rational comparisons may be made among alternative courses of action. Performance models are required in two distinct contexts, depending on the pavement management level involved. At the project level, fairly detailed and specific models are required for predicting the performance expected for an individual pavement section. At the network level, general or average prediction models are required to provide estimates of the expected performance for a typical pavement or class of pavements. Accordingly, quite distinct modeling methods are indicated for these two different modeling needs. Performance-modeling requirements and data requirements for both network-level and project-level applications are discussed. An idealized experiment to collect data for performance modeling is presented. A specific performance prediction model based on stochastic concepts and treating pavement deterioration as a Markov process is presented as an example of the development of prediction models for network-level applications.

All highway agencies are faced with the problem of providing and maintaining a network of roadways to serve the public. This requires both a considerable capital investment and an adequate maintenance and rehabilitation program. During the past decade, various economic, social, and political factors have made it increasingly important that transportation agencies take every step to make the most beneficial use of their often-inadequate budgets. This has resulted in the rise of the pavement management system from a theoretical concept discussed by university professors to a practical reality under development and implementation throughout the nation.

As a result of this increasing emphasis, both the conceptual and the practical elements of systematic pavement management have been widely discussed (1-11). Great strides have been made, but significant problems have also been encountered. One such problem, which will be addressed in this paper, is the difficulty in predicting pavement performance.

Systematic pavement management is based on the idea that it is possible to determine, in a reasonably objective fashion, how best to use the public funds made available for providing pavements. Budgets are typically allocated on a one- or two-year cycle, and construction, maintenance, and rehabilitation activities are generally planned on an annual basis. Nevertheless, activities carried out (or postponed) now can have a significant impact on roadway conditions for several years or even decades. In order to make rational choices among alternative courses of action, it is therefore necessary to be able to predict or estimate the future performance of the roadway under each alternative action.

Pavement performance has in the past generally been defined as a summary or accumulation of pavement serviceability index based on objective measurements of roughness and/or pavement distress. This use of the word "performance" stems from the work of Carey and Irick (12), although their original definition left considerable room for greater generality. More specifically, performance has been equated with the area under the serviceability history curve or the shape of the serviceability curve. This is the concept of performance adopted in this paper. It should be mentioned, however, that there has been no universal agreement on the definition of pavement performance. For example, in the recent literature, pavement performance is defined variously as (a) the ability of a pavement to provide an acceptable level of serviceability with a specified degree of reliability at an assumed level of maintenance (13), (b) allowable repetitions of loading prior to the functional failure of the pavement (14), and (c) the probability that a critical life of the pavement will be achieved based on the onset of critical conditions (15).

Since serviceability is almost universally measured by using a serviceability index based on roughness or riding comfort, the generally accepted use makes pavement performance a function of pavement roughness. However, many other factors, such as skid resistance, structural adequacy, and cracking, may be important in determining the overall adequacy of a pavement. The word "performance" is a natural candidate to describe this overall adequacy, so it is somewhat unfortunate that it has been defined more narrowly as a function of roughness. We are hopeful that at some future time pavement specialists can agree to reserve the word "performance" to denote this overall adequacy.

PERFORMANCE-MODEL REQUIREMENTS FOR PAVEMENT MANAGEMENT

Performance models are used in two distinct contexts as a part of pavement management, depending on the pavement management level involved. At the project level, fairly detailed and specific models are required for predicting the performance expected for an individual pavement section. At the network level, general or average prediction models are required to provide estimates of the expected performance for a typical pavement or class of pavements. Accordingly, quite distinct modeling methods are indicated for these two different modeling needs.

Project-Level Models

At the project level, considerable information will be available regarding the pavement structure, the current and expected traffic, current and past distress measurements, deflection, and so forth. The prediction model used must be able to predict specific values for the performance of the given section in an accurate and reliable fashion. Thus, a fairly accurate prediction model specific to the individual conditions appropriate to a single project is needed.

One approach to project-level modeling is based on the use of current and historical information on pavement condition to predict the future serviceability of the pavement. Such models are often termed "distress/performance relationships," and the problem of relating pavement distress to serviceability and performance has been under attack for some time (16-20). Time-dependent distress/performance relationships that are broadly applicable but that yield accurate predictions for individual sections of roadway are extremely difficult to derive. The primary reasons for this difficulty are the lack of adequate data records that cover a sufficiently long time period and the inherent variability associated with measurements of pavement condition.

A variant to the distress/performance problem involves predicting future distress and then relating distress to serviceability in a time-independent model. Distress-prediction models for various distress types are available and have been discussed and evaluated in the literature (13, 17, 21-27). At the project level, it is feasible to obtain sufficient data to provide input to one or more of these mechanistic models. The output would be a prediction of one or more future distress levels. It then only remains to relate these future distress levels to serviceability. We recently found that, given the current state of available data, it is in fact more feasible to relate distress to serviceability directly, with no time dependence, then to develop time-dependent relationships (17).

It should be mentioned that some of the mechanistic distress models referred to above (VESYS, for example) also include serviceability predictions. Thus, at the project level, some performance-prediction models that incorporate distress/performance relationships are already available.

Models developed from data collected on small groups of similar pavements are more likely to be reliable than those developed from large data bases. Therefore, by carefully selecting several classes of similar pavements, an agency could produce time-independent distress/serviceability relationships for each class that would be reliable enough for project-level pavement management use. Such models would, of necessity, be very limited in applicability; that is, each model would apply to only a very small class of pavements, so that each agency would require several such models in order to predict performance for a variety of pavement projects. The number of pavement sections to be included in each modeling class and the number of different classes to be used will depend on the needs and resources of the agency. In the extreme case, a separate model could be used for each pavement section. A single functional form that has variable coefficients could be chosen to represent the desired relationship for a wide variety of pavements, and the coefficients could be determined separately for each section of pavement. Such an approach requires access to considerable historical information for each section.

This same sort of approach can be employed without the use of mechanistic models to predict serviceability history or performance directly. When applied to individual pavement sections, this amounts to extrapolation of established performance trends, which again requires good records of past performance from which to extrapolate. Despite this requirement, at least one state highway agency has used this method with some success in predicting performance individually for thousands of pavement sections (28).

Network-Level Models

Direct performance prediction for individual pavement sections is also viable for network-level pavement management. In fact, the agency referred to above has used the performance predictions for individual sections for programming purposes. However, this method was only recently adopted after a decade of pavement management system development, data-base organization, and data collection. Previously, a subjectively based performance-prediction technique was used (4,29).

The other project-level modeling techniques discussed above are less viable at the network level. The mechanistic distress models require information of a character that is much too detailed for network-level applicatons. Even if such details were available, the amount of time required for the detailed analysis would be prohibitive. On the other hand, the formulation of direct, time-dependent distress/serviceability relationships is probably not feasible in the absence of a long-term data record. Thus, the development of direct distress/serviceability relationships for network-level pavement management is not likely to be feasible for a number of years for most agencies.

There is, however, an alternative approach that involves subjective modeling of pavement performance. Markovian or Bayesian techniques may be used to develop performance-prediction models that use distress/serviceability relationships only indirectly. Since only an average performance prediction for any pavement section is required at the network level, the lack of adequate data is not as troublesome as it is for project-level modeling. Bayesian or Markovian techniques are particularly applicable for this case, and in fact these techniques may be implemented in situations when little or no objective data are available. An example of network-level performance prediction based on purely indirect distress/serviceability relationships is presented in a subsequent section of this paper.

Data Requirements for Performance Modeling

As discussed in the previous section, the availability of pavement data records has a significant impact on pavement performance modeling. During the conduct of recent research (17), we had occasion to review selected pavement condition data records from a dozen state highway agencies, the AASHO Road Test, and the Brampton Test Road. Data from each of these sources were found to be inadequate for the development of reliable performance models for pavement management purposes. The major factors that contribute to this inadequacy are discussed below. Of course, not all data sources exhibited all the inadequacies listed below. In some cases, only a single factor was missing, whereas in others several factors contributed to the inadequacy. However, in no case did a single data source prove entirely adequate. The following major inadequacies were identified:

1. Inadequate time records: Many of the data records reviewed involve only one to three years of pavement distress and serviceability data. The pavements represented may have an average service life of 20 years, so that such a limited sample would hardly provide an adequate basis for life-cycle performance modeling.

2. Omission of key variables: There is very little agreement among state agencies as to which pavement variables are essential. For flexible and composite pavements, only rutting was recorded universally in the data examined. None of the rigid-pavement distress variables were reported universally. In many cases, significant distress variables were lumped together or combined into a single index whose value was reported in the data record. Most data sources reported a present serviceability index (PSI) on a scale of 0-5, but some reported only roughness or bump count. Several states reported that distress and serviceability data were not available on the same data field or simply were not available for the same pavement sections at all. In addition, very limited maintenance records were included in these data.

3. Lack of standardization of units: The only variable universally reported in the same units was rut depth, recorded in inches. Even so, the method for determining average rut depth varied among the data sources examined. Other distress variables were recorded in a variety of units. For example, various forms of cracking were reported in units of square feet per thousand square feet of pavement surface, linear feet per thousand square feet, square feet of area affected, total length of cracking, number of cracks per section, and by distress level in terms of severity and extent.

Given the wide variety of data sources examined, it is likely that the problems encountered here are common to the majority of existing data sources. That is, data inadequacy is a widespread problem. Therefore, it is felt that some guidance should be provided for future data-collection efforts. It should be emphasized that this discussion applies specifically to data collected for modeling purposes and not to data collected for routine inventory or other purposes.

Beyond just correcting the obvious deficiencies, the only way to assure that meaningful modeling will be achieved is to design an experiment or experiments to incorporate all the relevant factors. Consequently, an ideal experiment design was developed to provide the data necessary for effective performance modeling. It is not anticipated that this particular experiment will be performed, but it is felt that the considerations discussed here will provide guidance for future data-collection efforts. The discussion deals only with flexible pavements, for purposes of an example. However, the same basic considerations carry over to rigid pavements, and a similar design could be constructed for the case of rigid pavements.

IDEAL EXPERIMENT DESIGN

The first step in the design of an experiment to collect data for pavement performance modeling is to identify the dependent variables (y's) to be measured during the experiment. This list of important variables should probably include (a) distress, (b) serviceability, (c) deflection, and (d) skid resistance. Each of these basic variables may involve several subvariables. For example, distress for flexible pavements will probably involve rutting and fatigue cracking as well as possibly low-temperature cracking, bleeding, or other variables. It is desirable to limit this set of variables as much as possible without excluding important parameters.

The next step is to acknowledge the role of time as a split-plot factor and not as a dependent variable or as a covariate. This forces the investigator to obtain measurements throughout the entire experiment at fixed intervals of time for all treatment combinations, which eliminates the inadvertent confounding of time with particular treatment combinations and allows investigation of the interactions of time with all factors in the experiment. This probably is the single most important concept in the experiment design. Any departure from taking observations in a regularly scheduled time sequence will have confounding effects that cannot be completely accounted for in the analyses to follow. Of course, the shortcoming for any time-dependent variables is that the errors may be correlated, and this will hinder the development of good prediction models.

Next, a list of factors and levels to consider in the ideal experiment must be developed. An example is given below:

Category	Factor
Structural	A. Surface thickness
	B. Surface type
	C. Base thickness
	D. Base type
	E. Subgrade strength
Environment	F. Moisture
	G. Temperature
	H. Freeze or thaw cycle
Load	J. Traffic or vehicle passes
	K. Percentage of trucks or equivalent axle load
Miscellaneous	L. Construction variability
	M. Drainage
	N. Maintenance, preventive
	O. Maintenance, corrective
	P. Geometry

These factors are not supposed to be exhaustive or mandatory. However, it is felt that 15 factors would be sufficient to include all major influences on pavement performance. It may be desirable to delete some factors in this table, such as G or H, which overlap to some degree. Similarly, some factors may need further subdivision, such as M, which may require separate treatment of surface and subsurface drainage.

If only two levels were run for each of the 15 factors, a design that would allow estimation of all main effects and two-factor interactions (assuming that three-factor interactions are zero) is a 1/128 replication of the 15 factors in 8 blocks of 32 each. (Of course, 8 blocks may not be necessary, and this design simply represents an ideal estimate.) Such a design is given by the National Bureau of Standards as Plan 128.15.32 (30, pp. 70 and 72), a 1/128 replication of 15 factors in 8 blocks of 32 units each. The identity, block confounding, and blocks for this design are reproduced in Figure 1 (30). Note that an appropriate block structure must be chosen, which may require a relabeling of the factors. For example, if the interaction (ABD) within the environmental category were to be used in blocks, then moisture, temperature, and freeze or thaw cycle would be renamed A, B, and D, respectively.

The design of Figure 1 is good for two-level interactions. However, it is anticipated that at least three levels will be needed in most (if not all) factors in order to investigate curvature (deviation from straight-line behavior). If curvature is needed in all the factors, a composite design could be run that would require a total of 31 more treatment combinations. Since probably only three levels would be used for each factor, we could represent the center point as zero level and denote "low" by -1 and "high" by +1. By using this set of definitions for the levels, the 31 treatment combinations given in Figure 2 should be added to the

Figure 1. Ideal experiment design that uses 15 factors in 8 blocks.

Plan 128.15.32. 1/128 replication of 15 factors in S blocks of 32 units each.

Factors: A,B,C,D,E,F,G,H,J,K,L,M,N,O,P.

I = ABEGN = ACEFNP = BCFGP = DEFGO = ABDFNO = ACDGNOP = BCDEOP = ADHKO
= BDEGHKNO = CDEFHKNOP = ABCDFGHKOP = AEFGHK = BFHKN = CGHKNP
= ABCEHKP = BCHJNOP = ACEGHJOP = ABEFHJO = FGHJNO = BCDEFGHJNP = ACDFHJP
= ABDGHJ = DEHJN = ABCDJKNP = CDEGJKP = BDEFJK = ABFGJKN = ABCEFGJKNOP
= CFJKOP = BGJKO = AEJKNO = ABKLOP = EGKLNOP = BCEFKLNO = ACFGKLO
= ABDEFGKLP = DFKLNP = BCDGKLN = ACDEKL = BDHLP = ADEGHLNP = ABCDEFHLN
= CDFGHL = BEFGHLOP = AFHLNOP = ABCGHLNO = CEHLO = ACHJKLN = BCEGHJKL
= EFHJKLP = ABFGHJKLNP = ACDEFGHJKLNO = BCDFHJKLO = DGHJKLOP
= ABDEHJKLNOP = CDJLNO = ABCDEGJLO = ADEFJLOP = BDFGJLNOP = CEFGJLN
= ABCFJL = AGJLP = BEJLNP = CDGHJMO = ABCDEFHJMNO = ADEFGHJMNOP
= BDFHJMOP = CEFHJM = ABCFGHJMN = AHJMNP = BEGHJMP = ACGJKM = BCEJKMN
= EFGJKMNP = ABFJKMP = ACDEFJKMO = BCDFGJKMNO = DJKMNOP = ABDEGJKMOP
= BDGMNP = ADEMP = ABCDEFGM = CDFMN = BEFMNOP = AFGMOP = ABCMO
= CEGMNO = ABGHKMNOP = EHKMOP = BCEFGHKMO = ACFHKMNO = ABDEFHKMNP
= DFGHKMP = BCDHKM = ACDEGHKMN = ABCDGHJKLMP = CDEHJKLMNP
= BDEFGHJKLMN = ADFHJKLM = ABCEFHJKLMOP = CFGHJKLMNOP = BHJKLMNO
= AEGHJKLMNO = BCGJLMOP = ACEJLMNOP = ABEFGJLMNO = FJLMO = BCDEFJLMP
= ACDFGJLMNP = ABDJLMN = DEGJLM = ADGKLMNO = BDEKLMO = CDEFGKLMOP
= ABCDFKLMNOP = AEFKLMN = BFGKLM = CKLMP = ABCEGKLMNP = GHLMN
= ABEHLM = ACEFGHLMP = BCFHLMNP = DEFHLMNO = ABDFGHLMO = ACDHLMOP
= BCDEGHLMNOP.

Block confounding: ABD, ACF, BCDF, ABCE, CDE, BEE, ADEF

Blocks only: All two-factor interactions are measurable.

Blocks

1	2	3	4	5	6	7	8
(1)	bcdfgjmo	acdghk	abfhjkmo	acdgjl	abflmo	hjkl	bcdfghklmo
cefhjklmn	bdcghklno	adefgjlmn	abcelno	adcfghkmn	abcehjkno	cefmn	bdcgjno
adefhjlop	abceghlmp	cefgjklop	bdeklmp	cefghop	bdchjmp	adcfkop	abcegjkmp
acdkmnop	abfgjknp	ghmnop	bcdfhjnp	gjklmnop	bcdjklnp	acdhjlmnop	abfghlnp

Figure 2. Additional treatment combinations needed to investigate curvature in ideal experiment.

FACTOR

	A	B	C	D	E	F	G	H	J	K	L	M	N	O	P
1.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	+1
3.	0	0	0	0	0	0	0	0	0	0	0	0	0	0	-1
4.	0	0	0	0	0	0	0	0	0	0	0	0	0	+1	0
5.	0	0	0	0	0	0	0	0	0	0	0	0	0	-1	0
6.	0	0	0	0	0	0	0	0	0	0	0	0	+1	0	0
7.	0	0	0	0	0	0	0	0	0	0	0	0	-1	0	0
8.	0	0	0	0	0	0	0	0	0	0	0	+1	0	0	0
9.	0	0	0	0	0	0	0	0	0	0	0	-1	0	0	0
10.	0	0	0	0	0	0	0	0	0	0	+1	0	0	0	0
11.	0	0	0	0	0	0	0	0	0	0	-1	0	0	0	0
12.	0	0	0	0	0	0	0	0	0	+1	0	0	0	0	0
13.	0	0	0	0	0	0	0	0	0	-1	0	0	0	0	0
14.	0	0	0	0	0	0	0	0	+1	0	0	0	0	0	0
15.	0	0	0	0	0	0	0	0	-1	0	0	0	0	0	0
16.	0	0	0	0	0	0	0	+1	0	0	0	0	0	0	0
17.	0	0	0	0	0	0	0	-1	0	0	0	0	0	0	0
18.	0	0	0	0	0	0	+1	0	0	0	0	0	0	0	0
19.	0	0	0	0	0	0	-1	0	0	0	0	0	0	0	0
20.	0	0	0	0	0	+1	0	0	0	0	0	0	0	0	0
21.	0	0	0	0	0	-1	0	0	0	0	0	0	0	0	0
22.	0	0	0	0	+1	0	0	0	0	0	0	0	0	0	0
23.	0	0	0	0	-1	0	0	0	0	0	0	0	0	0	0
24.	0	0	0	+1	0	0	0	0	0	0	0	0	0	0	0
25.	0	0	0	-1	0	0	0	0	0	0	0	0	0	0	0
26.	0	0	+1	0	0	0	0	0	0	0	0	0	0	0	0
27.	0	0	-1	0	0	0	0	0	0	0	0	0	0	0	0
28.	0	+1	0	0	0	0	0	0	0	0	0	0	0	0	0
29.	0	-1	0	0	0	0	0	0	0	0	0	0	0	0	0
30.	+1	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31.	-1	0	0	0	0	0	0	0	0	0	0	0	0	0	0

256 in the original design, which would make 287 combinations. Of course, there would need to be repeats of these 287 treatment combinations. It would be ideal to have a complete replicate of the whole experiment, but one may conceive of 13 repeats if engineering information were available on the experimental error and the 13 were used only to check the error. This would yield a total of 300 treatment combinations.

Analysis

Let us assume that all measurements of serviceability (y_1), deflection (y_2), skid resistance (y_3), rutting (y_4), and fatigue cracking (y_5) have been taken over the time intervals desired for all 300 treatment combinations. There may be as many time intervals as desired, but an ideal experiment should encompass a reasonably large fraction of the estimated service life of the pavements.

Analysis at Each Time Period

One could run an analysis of variance (ANOVA) on the 256 treatment combinations plus the 13 repeats for pure error at each time period for each of the five y's (assuming that appropriate transformations were made to make all variables normally distributed). The ANOVA would involve the sources and degrees of freedom (df) identified below:

Source	df
Blocks	7
δ	0
Main effects (ME)	15
Two-factor interactions (2fi)	105
Residual	128
Pure error	13
Total	268

After finding out which two-factor interactions are significant, one could use all 300 observations and run a multiple regression on each dependent variable, y_i ($i = 1,2,\ldots,5$), as follows:

$$y_i = \beta_0 + \beta_1 x_1 + \ldots + \beta_{15} x_{15} + \beta_{1,1} x_1^2 + \ldots + \beta_{15,15} x_{15}^2$$
$$+ \text{ all two-factor interactions significant in ANOVA for}$$
$$\text{ith dependent variable} + \text{residual} + \text{pure error} \quad (1)$$

Analysis over Time

Here time is a split-plot factor. Run an ANOVA on each y, say, over 11 time periods, to get a number to show in the ANOVA (the number of time periods could be greater or smaller). The sources and df's of this ANOVA are shown below:

Source	df
Blocks	7
δ_1	0
ME (A-P)	15
2fi (A-P)	105
Residual	128
Pure error	13
δ_2	0
Time (T)	10
T x blocks	70
T x ME	150
T x 2fi	1050
T x residual	1280
T x pure error	130

The most important part of the ANOVA shown above is to find out whether the interpretation of T x pure-error mean square is of the same order of magnitude as pure-error mean square. This concept has been covered by Anderson and McLean (31, Chapter 7). The next important part, given that the first one shows that these errors are the same size, is T x residual versus residual, followed by (T x 2fi) versus two-factor interactions, and finally (T x ME) versus main effects.

If the errors (pure and residual) can all be pooled, then an overall regression analysis may be run for each y_i as follows:

$$y_i = \beta_1 x_1 + \ldots + \beta_{15} x_{15} + \beta_{1,1} x_{15}^2 + \ldots + \beta_{15,15} x_{15}^2$$
$$+ \text{ significant two-factor interactions of the 15 factors}$$
$$+ \text{ significant time and interactions of time effects}$$
$$+ \text{ residual} + \text{pure error} \qquad (2)$$

If it turns out that the errors (pure and T x pure) cannot be pooled, there may be correlation of the errors. To examine the effects of this condition in the factorial part, one may use the procedure given by Anderson and McLean (31, p. 166). In this case, one calculates the sums of squares as given above, but the df's for the sources are used as listed below:

Source	df
T	1
T x blocks	7
T x ME	15
T x 2fi	105
T x residual	128
T x error	13

If the results of all the F-tests are the same as for the previous tests by using 10 times the df's, one need not be concerned about correlated errors. If, however, there are major differences, care must be taken in the interpretation and use of the variables in the regression equations. There is no clear-cut way to obtain ideally all the information due to time if the errors are too highly correlated.

Other Design Approaches

There are many types of designed experiments that could be used for this problem. However, the most efficient one seems to be the one discussed above.

If it is necessary to investigate three-factor interactions, an entirely different design must be made, which requires many more treatment combinations than the design presented here. If curvature must be examined for all combinations of the 15 factors, a fractional factorial of 3^{15} may be needed. The number of treatment combinations required for this type of design is quite large.

Other designs could involve fewer factors if, for example, a state agency felt that some of the factors listed earlier were not needed to represent conditions that faced the agency adequately. However, the primary problem must still be faced: In order to develop models capable of accurately predicting the performance of a given section of pavement, considerable data must be available for that section (or similar sections) over a fairly long period of time. This data requirement may be partly circumvented through the use of subjective data or expert opinion. Subjective models based on Bayesian theory may be used for several years until adequate objective data can be acquired (32). In this approach the requirement for objective data is replaced by a similar requirement for experts who have had considerable experience in the performance of pavements over a long period of time. Most highway agencies have such knowledgeable people to draw on, so this approach offers great promise in future modeling applications.

STOCHASTIC SERVICEABILITY DETERIORATION MODEL FOR RIGID PAVEMENTS

Network-level pavement management requires performance predictions that are reliable on the average. That is, specific predictions for individual sections may deviate considerably from actual performance as long as random fluctuations are involved. Thus, stochastic models are particularly suited for network-level applications such as rehabilitation programming.

The development of such a performance model is discussed in this section. The model presented is quite simple in scope and concept, and it is not expected to be universally implemented. Rather, it is hoped that this discussion may provide interested agencies with guidance in the development of probabilistic performance models. The techniques employed have been discussed in the literature (27, 2, 32-38).

Development of Model

As part of a recent modeling effort (17), we examined serviceability histories for rigid-pavement sections from loops 3, 4, 5, and 6 of the AASHO Road Test. It was observed that roughly 7 of every 10 sections that reached terminal serviceability during the road test exhibited a characteristic serviceability pattern--a long period of nearly constant serviceability followed by a precipitous drop near the end of the service life. This pattern is evident in Figure 3, which illustrates serviceability plotted against service life for 20 of the rigid-pavement sections that failed during the road test. Service life is defined here as the length of time between the beginning of the test and the time at which a particular section reached terminal serviceability. Thus, the service-life scale used in the figure is a time scale normalized by the total length of time that each pavement section was in service.

The pattern illustrated in Figure 3 was found for pavement sections that had a slab thickness that ranged from 2.5 in to 11 in, applied axle loads that ranged from 6-kip single axles to 48-kip tandem axles, pumping scores from 500 to 60 000, and a similar range of other parameters. Thus, it was felt that this pattern could serve as the basis for a fairly general, widely applicable performance model.

There are a number of functional forms that could be used to reproduce the general shape illustrated in Figure 3. In the hope of obtaining a model that could be adapted to a variety of pavement types and structures, the following general form was chosen:

$$\overline{PSI}(T) = C_1 + \left(C_2 / \{\exp[\beta(T/\tau - 1)] + 1\}\right) \qquad (3)$$

where

$\overline{PSI}$ (T) = average predicted serviceability at time T;

C_1, C_2 = constants determined from initial and terminal serviceabilities;

β = parameter, presumably dependent on pavement structure, load, and environment, that determines the shape of the predicted serviceability history curve; and

τ = expected service life of pavement (time in years from beginning of traffic to terminal serviceability).

By adjusting the values of coefficients τ and β, Equation 3 can be made to reproduce the shape of a wide variety of serviceability patterns for flexible or rigid pavements. The values chosen here to reproduce the behavior of Figure 3 are shown in the following equation:

$$\overline{PSI}(T) = -1.5 + \left(6.0 / \{\exp[10(T/\tau - 1)] + 1\}\right) \qquad (4)$$

Figure 3. Serviceability history (normalized service-life time scale) for AASHO Road Test rigid pavements.

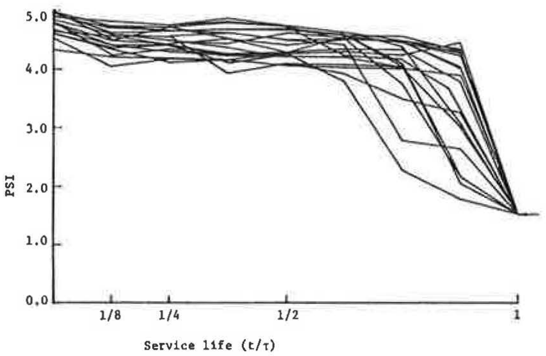
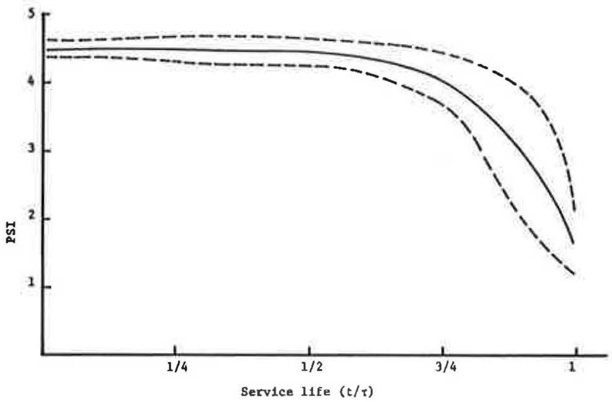


Figure 4. Predicted serviceability history based on stochastic version of Equation 4.



This equation is plotted as the solid curve in Figure 4. Note that the parameter τ need not be specified in order to compute serviceability at any fraction of the expected service life. However, the value of τ must be fixed in order to translate this fraction of service life into an actual elapsed time in years.

Equation 4 represents a very simple serviceability prediction model. Of course, some variability is observed in Figure 3 for the serviceability of individual pavement sections. In order to account for this, a stochastic feature was added to the prediction given by Equation 4. In this approach, the PSI predicted by Equation 4 is to be interpreted as a mean serviceability index for the pavements in question. Variations about this mean are incorporated by defining an artificial variance or standard deviation. Estimated values for such a standard deviation were derived from the magnitude of the variation observed in Figure 3. These values were used to construct the dashed lines in Figure 4, which represent the mean value plus or minus twice the artificial standard deviation.

In order to make practical use of this stochastic feature, the predicted serviceability history of Equation 4 and Figure 4 was incorporated in a Markovian framework. In such an approach, the pavement is described as being in a certain state at any given time, and the probability that the pavement will undergo a transition to each other possible state within a fixed short period of time is specified. Such a model is conveniently expressed in matrix notation--transition probabilities are arrayed in a square matrix, and possible pavement states are listed in a single-column matrix. In this example, pavement states are specified in terms of pavement serviceability, but other significant variables may be incorporated in the description of pavement states (37, 38).

Example

Twenty possible pavement states, or serviceability values, were selected for use in this example. These are listed below:

State	Nominal PSI	State	Nominal PSI
1	5.0	11	3.8
2	4.9	12	3.6
3	4.8	13	3.4
4	4.7	14	3.2
5	4.6	15	3.0
6	4.5	16	2.7
7	4.4	17	2.4
8	4.3	18	2.1
9	4.2	19	1.8
10	4.0	20	1.5

Nominal serviceability values are specified for each state. Once these states have been specified, transition probabilities between states may be calculated by using Equation 4 and Figure 4. A set of transition probabilities that effectively reproduce the behavior illustrated in Figure 4 are shown in Figure 5. In developing this transition matrix, the time interval between transitions was fixed at 1/100 of the expected service life of the pavement. Thus, for a service life of 20 years, five transitions per year are incorporated in Figure 5. At each transition, the pavement may remain in its current state or enter another state (improve or deteriorate).

Predictions of pavement serviceability are carried out in the following manner. First, the initial state of the pavement is specified. This is done in terms of the probability that the pavement has a specific serviceability level at the initial time. If the pavement is known to have a PSI of 4.5 exactly, then the probability that the pavement exists in state number 6 given above is 1.0, and the probability for all the other states is 0. However, in the general case, the serviceability of the pavement can be specified only within some limit, say, a mean serviceability of 4.5 and standard deviation 0.1. The initial state specification for this case is given in Table 1. Such a state is called a mixed state. The probability values in this case may be thought of as expressing the likelihood that a repeat measurement of PSI would yield the nominal PSI value asociated with each state in the table.

The state of the pavement at future times is calculated by multiplying the initial state by the transition matrix. The state of the pavement after one transition is obtained by multiplying the initial state by the transition matrix once. For two transitions, the multiplication is carried out twice, and so forth. In this example, the state of the pavement at the midpoint of its service life would require 50 such multiplications. Thus, in actual practice, it might be advisable to use fewer transitions per year, perform the calculations on a computer, or both.

In this approach, the procedure for obtaining predictions for future PSI values is fixed: Multiply the existing pavement state by the transition matrix. However, the formalism allows modification of pavement states and transition probabilities to account for such effects as resurfacing, accelerated pavement distress, increased traffic, and so forth. If the observed state of the pavement is found to

Figure 5. Transition probability matrix for Marcov sample problem.

TRANSITIONS TO THESE PSI STATES

TRANSITIONS FROM THESE PSI STATES	5.0	4.9	4.8	4.7	4.6	4.5	4.4	4.3	4.2	4.0	3.8	3.6	3.4	3.2	3.0	2.7	2.4	2.1	1.8	1.5
5.0			1.0																	
4.9			1.0																	
4.8			.010	.030	.100	.220	.280	.220	.120	.020										
4.7			.002	.026	.153	.448	.210	.095	.053	.013	.001									
4.6			.002	.024	.145	.421	.202	.099	.074	.030	.004									
4.5			.002	.023	.137	.399	.193	.097	.081	.049	.017	.003	.001							
4.4			.002	.023	.137	.399	.193	.097	.081	.049	.017	.003	.001							
4.3			.002	.021	.127	.371	.179	.091	.079	.058	.036	.021	.011	.004	.001					
4.2			.002	.021	.119	.272	.184	.108	.098	.074	.048	.034	.022	.012	.006	.001				
4.0			.003	.012	.042	.097	.139	.138	.150	.120	.082	.063	.052	.038	.032	.019	.008	.003	.001	
3.8				.002	.011	.030	.061	.088	.153	.159	.116	.090	.078	.061	.059	.045	.026	.013	.006	.003
3.6						.002	.006	.017	.055	.134	.148	.128	.112	.091	.094	.083	.058	.036	.021	.016
3.4						.002	.005	.014	.045	.108	.119	.103	.092	.077	.084	.084	.073	.060	.053	.085
3.2								.003	.015	.056	.096	.111	.107	.091	.100	.100	.087	.072	.060	.102
3.0								.001	.006	.024	.056	.089	.109	.106	.117	.118	.102	.085	.070	.119
2.7									.001	.011	.029	.058	.091	.107	.130	.137	.119	.098	.082	.139
2.4										.005	.015	.032	.066	.089	.130	.153	.139	.115	.096	.162
2.1										.002	.008	.017	.044	.069	.112	.157	.156	.134	.112	.190
1.8										.002	.008	.017	.044	.069	.112	.157	.156	.134	.112	.190
1.5											.002	.007	.024	.045	.090	.147	.164	.154	.133	.233

Table 1. Initial state specification for PSI = 4.5, SD = 0.1.

State	Nominal PSI	Probability	State	Nominal PSI	Probability
1	5.0		11	3.8	
2	4.9	0	12	3.6	
3	4.8	0.006	13	3.4	
4	4.7	0.061	14	3.2	
5	4.6	0.241	15	3.0	
6	4.5	0.383	16	2.7	
7	4.4	0.241	17	2.4	
8	4.3	0.061	18	2.1	
9	4.2	0.007	19	1.8	
10	4.0	0	20	1.5	

differ from the predicted state, then the observed state may be substituted into the matrix multiplication process. Such a difference in observation and prediction could occur, for example, if the pavement were resurfaced after, say, two-thirds of the expected service life.

Of course, resurfacing, the occurrence of accelerated distress, or a dramatic increase in traffic volume could affect the expected rate of pavement deterioration as well as the current state of the pavement. These effects may also be incorporated into this formalism by adjusting the transition probabilities or by replacing Figure 5 with a transition matrix calculated on the basis of a faster or slower rate of deterioration. Thus, one transition matrix could be specified for the original pavement, another for overlaid pavements, etc. This is the approach taken by Smith (38). If several different transition matrices are required for each pavement section to be studied, quite a large number of calculations would be required. However, the behavior illustrated in Figure 3 indicates that a wide variety of pavements may be represented by a single matrix. Thus, an agency could have one transition matrix, say, for each functional class of new pavement. An additional matrix could be specified for each functional class for overlaid pavements. If the agency must deal with pavements in widely differing environments, then a different set of matrices could be required for each different region. Thus, there is a reasonable expectation that 20 or 30 transition matrices could be sufficient to provide serviceability predictions for most or all of the pavements for which an agency is concerned. Such predictions would of necessity represent the average expected serviceability pattern for the pavement functional class, environment, etc., rather than the best estimates for an individual pavement section. Hence, such an approach is expected to be most useful for network-level pavement management applications.

SUMMARY

Pavement performance modeling is an essential part of good pavement management, and at the same time it is a very complex task. In general, the development of good performance models requires a good long-term data base, and the ideal experiment suggested here can provide guidance to agencies that wish to acquire such a data base. However, the acquisition of relevant data is of necessity a long-term operation, so it is important to realize that useful performance models may be developed for more immediate application.

The best approach to development of short-term performance models depends on the intended use of the model. At the project level, the use of existing mechanistic pavement distress models along with time-independent distress/performance correlations developed for small groups of similar pavements may provide acceptable performance models. At the network level, less detail is desired, and models based on probabilistic concepts and expert opinion may be acceptable. The simple stochastic model presented here is an example of the application of probabilistic concepts to network-level performance modeling that may provide guidance for agencies that wish to take such an approach.

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REFERENCES

1. W.R. Hudson, R. Haas, and R.D. Pedigo. Pave-

ment Management System Development, NCHRP, Rept. 215, 1979.
2. R. Haas and W.R. Hudson. Pavement Management Systems. McGraw-Hill, New York, 1978.
3. Pavement Management Guide. Roads and Transportation Association of Canada, Ottawa, 1977.
4. R.L. Terrel and R.V. LeClerc. Pavement Management Workshop, Tumwater, Washington. FHWA, U.S. Department of Transportation, Rept. FHWA-TS-79-206, Sept. 1978.
5. R.K. Kher and W.A. Phang. OPAC Design System. Proc., 4th International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.
6. R.L. Lytton, W.F. McFarland, and D.L. Schafer. Flexible Pavement Design and Management Systems Approach Implementation. NCHRP, Rept. 160, 1975.
7. B.F. McCullough and D.R. Luhr. Pavement Design and Management System for Forest Service Roads--Implementation. Council for Advanced Transportation Studies, Univ. of Texas at Austin, Res. Rept. 60, 1979.
8. J.P. Mahoney, N. Ahmed, and R.L. Lytton. Optimization of Pavement Rehabilitation and Maintenance by Use of Integer Programming. TRB, Transportation Research Record 674, 1978, pp. 15-22.
9. M.Y. Shahin and F.M. Rozanski. Automated Pavement Maintenance and Repair Management System. U.S. Army Construction Engineering Research Laboratory, Champaign, IL, Interim Rept. CERL-IR-C-79, June 1977.
10. F.N. Finn, R. Kulkarni, and J. McMorran. Development of a Framework for a Pavement Management System. Woodward-Clyde Consultants, San Francisco; Arizona Department of Transportation, Phoenix, Final Rept., Aug. 1976.
11. W.R. Hudson and B.F. McCullough. Flexible Pavement Design and Management Systems Formulations. NCHRP, Rept. 139, 1973.
12. W.N. Carey and P.E. Irick. Pavement Serviceability-Performance Concept. HRB, Bull. 250, 1960, pp. 40-58.
13. W.J. Kenis. Predictive Design Procedures--VESYS User's Manual. FHWA, U.S. Department of Transportation, Final Rept. FHWA-RD-77-154, Jan. 1978.
14. J.E. Crawford and M.G. Katona. State-of-the-Art for Prediction of Pavement Response. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Rept. FAA-RD-75-183, Sept. 1975.
15. C.K. Kennedy and N.W. Lister. Prediction of Pavement Performance and the Design of Overlays. U.K. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Rept. 833, 1978.
16. Structural Design of Asphalt Concrete Pavement Systems. TRB, Special Rept. 126, 1971.
17. W.R. Hudson, F.N. Finn, R.D. Pedigo, and F.L. Roberts. Relating Pavement Distress to Serviceability and Performance. FHWA, U.S. Department of Transportation, Final Rept. FHWA-RD-80-098, July 1980.
18. K.W. Smeaton. Interactive Approach to Developing Pavement Distress-Performance Relationships. Univ. of Waterloo, Waterloo, Canada, thesis, 1978.
19. M.W. Witczak and K.R. Bell. Remaining-Life Analysis of Flexible Pavements. Proc., AAPT, Vol. 47, Feb. 1978.
20. R.P. Smith and B.F. McCullough. Use of Condition Surveys, Profile Studies, and Maintenance Studies in Relating Pavement Distress to Pavement Performance. Center for Highway Research, Univ. of Texas at Austin, Res. Rept. 123-19, 1974.
21. J.B. Rauhut, F.L. Roberts, and T.W. Kennedy. Models and Significant Material Properties for Predicting Distress in Zero-Maintenance Pavements. FHWA, U.S. Department of Transportation, Interim Rept. FHWA-RD-78-84, June 1978.
22. R.C.G. Haas. Method for Designing Asphalt Pavements to Minimize Low-Temperature Shrinkage Cracking. Asphalt Institute, College Park, MD, Res. Rept. 73-1, Jan. 1973.
23. F.N. Finn, C. Saraf, R. Kulkarni, K. Nair, W. Smith, and A. Abdullah. Development of Pavement Structural Subsystems. NCHRP, Project 1-10B, Final Rept., Feb. 1977.
24. J.C.M. Ma. CRCP-2, An Improved Computer Program for the Analysis of Continuously Reinforced Concrete Pavement. Univ. of Texas at Austin, master's thesis, Aug. 1977.
25. M.W. Witczak. Pavement Performance Models, Volume I: Repeated Load Fracture of Pavement Systems. U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, Rept. FAA-RD-75-227-I, Aug. 1976.
26. F.R.P. Meyer, A. Cheetham, and R.C.G. Haas. Coordinated Method for Structural Distress Prediction in Asphalt Pavements. Presented at Annual Meeting, AAPT, Lake Buena Vista, FL, Feb. 1978.
27. F. Moavenzadeh and B. Brademeyer. Stochastic Model for Pavement Performance and Management. Proc., 4th International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, Aug. 1977.
28. R.V. LeClerc. State of the Practice. Presented at National Workshop on Pavement Management, Phoeniz, AZ, May 19-21, 1980.
29. F.N. Finn, R. Kulkarni, and K. Nair. Pavement Management System Feasibility Study. Washington Highway Commission, Olympia; Materials Research and Development, Oakland, CA; Final Rept. Aug. 1974.
30. Fractional Factorial Experiment Designs for Factors at Two Levels. Statistical Engineering Laboratory, National Bureau of Standards, Applied Mathematics Series 48, 1957.
31. V.L. Anderson and R.A. McLean. Design of Experiments: A Realistic Approach. Marcel Dekker, New York, 1974.
32. W.S. Smith, F. Finn, R. Kulkarni, C. Saraf, and K. Nair. Bayesian Methodology for Verifying Recommendations to Minimize Asphalt Pavement Distress. NCHRP, Rept. 213, June 1979.
33. F.S. Hillier and G.J. Lieberman. Introduction to Operations Research. Holden-Day, San Francisco, 1967.
34. J.J. Martin. Bayesian Decision Problems and Markov Chains. Wiley, New York, 1967.
35. R.M. Stark and R.L. Nicholls. Mathematical Foundations for Design: Civil Engineering Systems. McGraw-Hill, New York, 1972.
36. W.S. Smith, F.N. Finn, C. Saraf, and R. Kulkarni. Bayesian Approach to Prediction of Pavement Distress. TRB, Transportation Research Record 671, 1978, pp. 54-56.
37. M.A. Karan. Municipal Pavement Management System. Univ. of Waterloo, Waterloo, Canada, thesis, 1977.
38. W.S. Smith. Flexible Pavement Maintenance Management System. Univ. of California, Berkeley, dissertation, 1974.

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Illustration of Pavement Management: From Data Inventory to Priority Analysis

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A typical case history of a pavement management implementation project is summarized. The project covered a major part of the highway system of the province of Prince Edward Island (PEI) and was undertaken in conjunction with the Federal-Provincial Atlantic Provinces Highway Strengthening Improvement Agreement. An extensive field inventory was performed on approximately 500 km of PEI's Trans Canada and major-arterial road networks. These field tests provided the basis for detailed analysis to determine the pavement-improvement needs for the portion of the road network considered over a 10-year programming period. A program of pavement-improvement priorities over the programming period was then determined through economic analysis and an optimization of projects, improvement strategies, and project timing. The effect of budget level was then analyzed by comparing the expected average network serviceability profiles over the programming period at the specified budget level with the case of zero capital budget for both the Trans Canada and major-arterial networks. PEI is one of the first areas in North America to successfully adopt such a comprehensive pavement management system. This indicates that the concepts of pavement management, which have been developed during the last decade, are now being put into practice.

In 1978, Prince Edward Island (PEI) entered into an agreement with Transport Canada to strengthen sections of the Trans Canada Highway. Transport Canada carried out its own cost/benefit analysis, by which the needs of the province were determined. The strengthening then began on a cost-shared basis.

After the problem had been given due consideration, it was decided that the province's future needs would best be served by implementing a comprehensive pavement management system. This exercise, it was thought, would not only generate an adequate objective data base for determining pavement-improvement needs but would also ensure that the province's large investment in the pavement portion of its road network would be protected and that the traveling public would be provided with an adequate level of service.

The pavement management concept was first conceived in the mid-1960s to organize and coordinate the activities involved in achieving the best value possible for the available public funds. Since then, both the public and the private sectors have made extensive efforts in the development of pavement management technology. The purpose has been to provide highway engineers with the tools needed to manage their pavement networks more effectively.

The PEI Department of Highways, like many other highway agencies in North America, has always been in competition with other departments for the limited tax dollars available for public projects. In most instances, these other competing departments have been able to document their cases more effectively and convincingly than the highway department has; the main reason is that the highway department has lacked a systematic and objective approach for determining pavement-improvement needs, establishing priorities for these needs, and clearly illustrating the consequences of failing to implement these priorities.

In the light of these considerations, PEI decided to implement a pavement management system on its highway network. Consequently, a contract was awarded for implementing such a system, initially on approximately 500 km of the province's Trans Canada Highway and major-arterial road system. The remainder of the arterial network will be covered in the next phase of the project.

Although PEI is one of the smallest provinces or states in North America, its highway network provides an excellent self-contained proving ground for pavement management implementation. Moreover, all the elements exist for their network as they do for a much larger network of a big state or province and there is no need to select only a small pilot portion of a much larger network to test the system.

The project started in the fall of 1978 and was completed in 1979. The purpose of this paper is to describe the results of the project, specifically the following:

1. Field inventory scheme used to collect data for establishing the current status of the road network in terms of its surface condition, ride quality, and structural capacity;
2. Procedure used to identify needs for pavement improvements;
3. Evaluation of the rehabilitation alternatives considered for those roads that require immediate action and/or will require rehabilitation within the next 10 years;
4. Priority-analysis technique used, which is based on objective field measurements, detailed economic analysis, and specified budget constraints; and
5. Budget-level analysis used to test the effects of different budget levels on the annual average condition of the network.

FIELD MEASUREMENTS

The program for acquiring field inventory information on the 500.9 km of road involved (a) section identification, (b) deflection measurements, (c) roughness measurements, (d) condition survey, and (e) core sampling.

Section Identification

In the first phase of the field work, 500.9 km of road were divided into sections. Highway department personnel provided the necessary input. Past experience, contract length, and the following factors were considered in the section-identification process: traffic volumes; pavement type, age, and thickness; and geometric characteristics (number of lanes, length, etc.). An attempt was made to identify homogeneous sections on the basis of traffic volume, pavement type and thickness, and geometric characteristics.

Extensive field studies and discussions with department personnel produced 25 sections on the Trans Canada Highway and 73 sections on other routes, which resulted in a total of 98 sections. A detailed list of the sections included in the project are given in a report prepared by Pavement Management Systems (1).

Deflection Measurements

In the spring of 1979, deflection measurements were taken in the outer wheelpath of each road section by using a Dynaflect unit. An average of six tests per kilometer was taken.

Roughness Measurements

The automatic road-analyzer (ARAN) unit was used in the summer of 1979 to measure road surface roughness on one full lane length of each road section. This involved sampling at 50-m intervals at a travel speed of 50 km/h.

The ARAN unit (2,3), which has been used extensively across North America, has the capability of simultaneously obtaining data on surface roughness and condition, skid resistance, grade angle, and transverse profile (including rut depth and cross slope) at normal travel speeds on an automated basis. This unit is housed in a Ford van and measures roughness by use of an accelerometer. The data are recorded in digital form. The unit has an on-board intelligent computer terminal that has a keyboard plus an acoustic coupler transmission system. It also has hard-copy recording and on-board editing capabilities by using specially developed software. Extensive repeatability measurements at various speeds and roughness levels have been made on the unit in cooperation with the Ministry of Transportation and Communications of Ontario. These indicated a high degree of repeatability and the fact that the unit can be used over a wide speed range (i.e., can "float" in traffic).

Condition Survey

The surface condition of each road section was also rated by evaluating and recording various types of surface defects or distress on a specially developed keyboard in the ARAN unit. Each type of distress, the extent, and the severity were actually recorded in coded form on the keyboard.

Core Sampling

One core per kilometer or a minimum of one core per section was taken to determine the layer thicknesses of the existing pavements. Subgrade soil type and the condition of the materials in the existing pavement structure were also determined by inspection in the field.

ANALYSIS OF DATA

Structural-Adequacy Analysis

The deflection readings taken on each road section were first adjusted to a spring value by multiplying by a spring/fall ratio. The mean, standard deviation, and design deflection ($\bar{x}$ + 2s) for the adjusted deflection measurements were then calculated for each road section.

These adjusted deflection measurements were then used with the appropriate traffic data (supplied by the department) to determine the structural-adequacy rating (SAR) of each pavement section. The SAR for a pavement section reflects the degree of structural deficiency, if any, that exists in the pavement structure. (SAR values that range between 0 and 4.9 indicate structural inadequacy, whereas values between 5.0 and 10.0 indicate structural adequacy.) Pavement Management Systems has detailed descriptions of the method used to calculate the SAR (1,4). Table 1 presents 1979 SAR values for all road sections analyzed in the project.

Riding-Comfort-Index Analysis

The ARAN roughness data collected on each road section were first edited to check for errors that might have occurred in the field. High roughness readings due to extreme external effects (i.e., railway crossings, etc.) were eliminated from the data.

Speed corrections were made to convert readings at actual test speeds to roughness at 50 km/h. A mean roughness value was then calculated for each section. The mean ARAN roughness value for each section was converted to a riding-comfort-index (RCI) value by using a previously established correlation between ARAN roughness and subjective panel ratings. This relationship, which was developed specifically for PEI conditions, can be found in a Pavement Management Systems report (5). Table 1 presents 1979 RCI values for all road sections analyzed in the project. The RCI ranges between 0 and 10; a value of 0 indicates the worst imaginable riding comfort, whereas a value of 10 indicates a perfectly smooth pavement (6).

Condition-Index Analysis

The condition survey data collected on each road section were edited at the same time as were the ARAN roughness data. The codes assigned to the various types, amounts, and severities of distress surveyed were then translated into numerical values. These condition indices (CIs) varied between 0 and 10. A value of 10 indicates no observable amount of a particular type of distress, whereas a value of 0 indicates very severe and extensive amounts of a particular type of distress.

The index values assigned to the 10 different distress types were then weighted according to the importance of each type of distress and combined into one CI. (See Table 1 for the average CIs calculated for all pavement sections analyzed in the project.)

Overall-Serviceability Analysis

In this project, the overall serviceability of a pavement section was assumed to be a function of its structural adequacy and riding comfort. The overall serviceability index (SI) is expressed as follows:

$$SI = a(RCI) + b(SAR) \quad (1)$$

where the overall SI is measured on a scale of 0-10, on which 10 represents a perfectly smooth and strong pavement and 0 represents a totally unacceptable pavement, and a and b are weighting factors (a + b = 1.0).

The weighting factors generally depend on the agency's policy and objectives. Some agencies give more weight to the user by assigning a higher weighting factor (a) to the RCI. Similarly, the structural characteristics may be more important to other agencies. This can be reflected in the calculations of overall SI by using a higher weighting factor (b) for SAR.

In PEI, different levels of weighting factors were used for different levels of RCI. The weighting factors used in the project for four levels of RCI and SAR are given below. These reflect the relative importance of RCI for very rough pavements and, conversely, the relative importance of SAR for smoother (but possibly weaker) pavements.

	Weighting Factor	
RCI Level	a (for RCI)	b (for SAR)
<4.0	1.0	0.0
4.1 - 6.0	0.6	0.4
6.1 - 8.0	0.4	0.6
>8.0	0.0	1.0

The SI of each road section was then calculated from Equation 1 and Table 2:

RCI = 5.3, SAR = 7.1: SI = 0.6 x 5.3 + 0.4 x 7.1 = 6.0
RCI = 3.5, SAR = 5.0: SI = 1.0 x 3.5 + 0.0 x 5.0 = 3.5.
RCI = 8.5, SAR = 6.2: SI = 0.0 x 8.5 + 1.0 x 6.2 = 6.2.

(See Table 1 for 1979 values of SI for all road sections analyzed in the project.)

Minimum Acceptable SI

Minimum acceptable SIs of 5.0 and 4.5 were used in the analysis for Trans Canada and major-arterial sections, respectively. Hence, a section on the Trans Canada Highway that had a present SI (PSI) of 5.0 or less was identified as requiring a major rehabilitation in 1979. Similarly, a Trans Canada section that had an acceptable SI in 1979 may have become a candidate project for rehabilitation if its predicted SI dropped to 5.0 or less within the 10-year programming period.

INPUT DATA TO PROGRAM

The types of input data obtained from the department were as follows: traffic information, alternative rehabilitation strategies, cost information, and budget information.

Traffic Information

The traffic data obtained for each section included average annual daily traffic (AADT), percentage of annual growth, and percentage of commercial traffic. The information was provided by the department's traffic personnel and was used directly in the project without any modifications. The most-recent traffic counts were employed in the AADT calculations. An AADT estimate was provided by the department for those sections that had no traffic counts.

Available Rehabilitation Strategies and Costs

The following five alternative rehabilitation strategies for each road section considered in the project were analyzed by the priority program: overlay type 1, overlay type 2, overlay type 3, reconstruction, and surface treatment. For PEI conditions, these were defined as follows:

1. Overlay type 1: Asphalt concrete overlay 39 mm thick placed on top of the existing pavement at a cost of $\$2.75/m^2$;

Table 1. Summary of results of sectional inventory analysis.

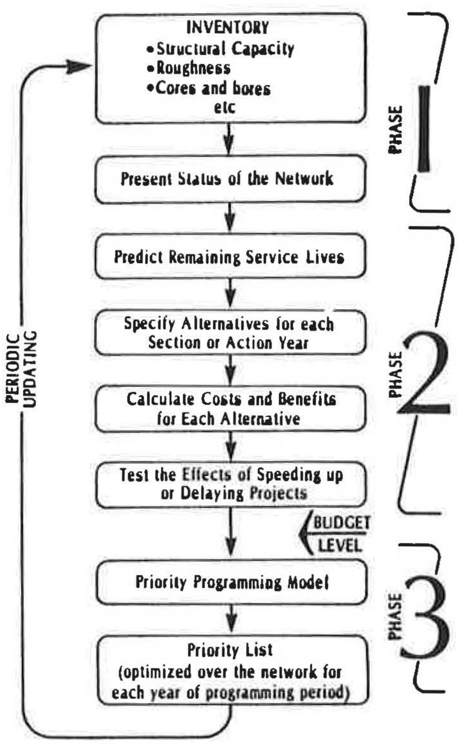
Section	CI	RCI	SAR	SI	Section	CI	RCI	SAR	SI
0001	2.8	2.6	0.5	2.6	0050	8.0	7.4	7.5	7.5
0002	3.8	3.0	0.8	3.0	0051	8.4	7.5	8.0	7.8
0003	4.8	2.8	0.5	2.8	0052	8.2	8.2	5.8	5.8
0004	3.3	4.9	2.3	3.9	0053	8.5	7.7	4.3	5.7
0005	3.0	3.7	1.0	3.7	0054	4.3	5.1	4.3	4.8
0006	4.2	5.1	0.0	3.1	0055	10.0	5.0	2.0	3.8
0007	4.2	5.4	1.3	3.8	0056	6.3	6.0	6.3	6.1
0008	4.2	4.7	0.5	3.0	0057	4.1	5.0	5.0	5.0
0009	3.7	6.8	0.8	3.2	0058	8.2	5.5	6.3	5.8
0010	7.2	6.5	3.5	4.7	0059	10.0	7.8	6.3	6.9
0011	7.9	6.8	0.0	2.7	0060	10.0	7.7	7.0	7.3
0012	6.6	6.7	0.0	2.7	0061	10.0	7.8	6.3	6.9
0013	5.7	4.2	0.0	2.5	0062	9.9	8.4	5.3	5.3
0014	4.1	4.1	1.5	3.1	0063	2.8	2.9	5.0	2.9
0015	6.4	4.9	1.8	3.7	0064	10.0	8.6	5.0	5.0
0016	4.4	4.1	0.0	2.5	0065	10.0	7.7	1.5	4.0
0017	4.0	5.4	1.0	3.6	0066	10.0	8.1	7.8	7.8
0018	2.1	2.9	3.3	2.9	0067	10.0	8.1	4.8	4.8
0019	5.9	3.0	3.0	3.0	0068	10.0	5.4	5.5	5.4
0020	9.7	7.9	4.3	5.7	0069	10.0	5.3	4.8	5.1
0021	4.2	4.6	1.8	3.5	0071	6.2	6.5	4.3	5.2
0022	7.6	1.8	0.8	1.8	0072	3.1	3.4	1.8	3.4
0023	3.6	3.7	4.0	3.7	0073	3.5	5.4	4.8	5.2
0024	9.6	7.3	5.0	5.9	0074	6.1	5.6	0.8	3.7
0025	3.6	3.5	3.5	3.5	0075	3.7	4.6	4.3	4.5
0026	2.7	3.8	2.0	3.8	0076	4.1	5.3	0.3	3.3
0027	5.7	4.3	0.5	2.8	0077	5.7	3.9	0.0	3.9
0028	4.8	4.0	0.8	4.0	0078	5.5	6.3	4.3	5.1
0029	7.3	4.4	4.8	4.6	0079	4.8	5.9	1.0	3.9
0030	7.2	4.9	5.0	4.9	0080	5.3	6.8	4.8	5.6
0031	5.4	6.0	5.0	5.6	0081	4.3	5.6	0.5	3.6
0032	6.0	6.9	4.5	5.5	0082	7.6	6.0	0.3	3.7
0033	5.0	7.3	3.5	5.0	0083	4.9	4.5	2.8	3.8
0034	6.3	7.4	2.3	4.3	0084	7.3	4.1	0.5	2.7
0035	6.0	7.4	1.3	3.7	0085	9.8	7.7	1.0	3.7
0036	4.0	6.2	0.8	3.0	0086	6.8	4.9	0.5	3.1
0037	6.3	7.5	0.8	3.5	0087	6.2	5.6	1.0	3.8
0038	10.0	7.3	2.0	4.1	0088	7.6	5.0	0.5	3.2
0039	10.0	7.4	1.5	3.9	0089	4.2	3.2	0.8	3.2
0040	10.0	7.9	5.0	6.2	0090	4.5	3.6	0.5	3.6
0041	10.0	8.1	6.5	6.5	0091	3.9	4.1	0.5	2.7
0042	7.6	5.9	1.5	4.1	0092	4.2	3.8	1.3	3.8
0043	9.6	7.0	4.5	5.5	0093	2.6	3.2	1.3	3.2
0044	8.8	7.4	2.5	4.5	0094	1.7	2.5	1.5	2.5
0045	3.6	5.9	2.3	4.5	0095	2.0	2.2	1.0	2.2
0046	4.5	5.7	5.0	5.4	0701	5.2	3.0	1.0	3.0
0047	7.0	6.3	3.8	4.8	0702	4.5	1.9	3.5	1.9
0048	6.6	4.7	1.3	3.3	0703	7.1	2.4	8.0	2.4
0049	7.2	4.1	1.5	3.1	0704	5.7	3.1	8.5	3.1

Note: CI = condition index; RCI = riding-comfort index; SAR = structural-adequacy rating; SI = serviceability index.

Table 2. Pavement rehabilitation improvement priorities by section over 10-year programming period (Trans Canada sections).

Year	Section Number	Type of Rehabilitation	Percentage of Rehabilitation
1979	55	A4	97
	63	A4	-
	701	A4	-
	702	A4	-
	703	A4	-
	704	A4	-
1980	54	A4	54
	55	A4	3
	57	A4	-
	64	A4	20
	65	A4	-
1981	54	A4	46
	56	A3	-
	61	A3	-
1982	58	A4	-
	62	A4	92
	64	A4	80
	67	A4	-
1983	52	A4	57
	62	A4	8
	71	A4	-
1984	60	A3	-
	66	A3	-
	68	A4	83
1985	52	A4	13
	53	A4	-
	68	A4	17
	69	A4	-
1986	52	A4	30
	59	A4	-
1987	-	-	-
1988	-	-	-

Figure 1. General structure of system.



2. Overlay type 2: Asphalt concrete overlay 89 mm thick placed on top of the existing pavement at a cost of $\$6.40/m^2$;

3. Overlay type 3: Asphalt concrete overlay 140 mm thick placed on top of the existing pavement at a cost of $\$10.03/m^2$;

4. Reconstruction: Completely new pavement (including paved shoulders on Routes 1, 1A, 2, 3, and 4) that has a structural design of 152-mm asphalt concrete surface and 254- to 305-mm base (existing in situ surface material is used as base after pulverization); costs of reconstruction were \$18.81, \$16.72, and $\$16.30/m^2$ for Trans Canada sections, major-arterial sections, and shoulders, respectively; and

5. Surface treatment: Double 9.5-mm chip seal that cost $\$2.00/m^2$.

Budget Information

An expected rehabilitation budget of \$3 600 000 was suggested by the department for the 119.4 km of Trans Canada sections considered in the project. Similarly, \$860 600 was assumed to be allocated to the sections on the major-arterial road network in the province. Although these figures were suggested specifically for 1980, they were assumed to remain constant (i.e., inflation was not included in budgets and costs) over the 10-year programming period.

NETWORK MANAGEMENT SYSTEM

The network management system described by Karan and Haas (7-9) is basically composed of three main phases (see Figure 1).

It starts with inventory work on the identified sections. Deflection measurements for determining SAR values, used to identify ride quality, and coring and boring constitute the major elements of this inventory.

The present status of the network is then determined from these inventory data. The PSI of each section in the network is calculated and will be described subsequently.

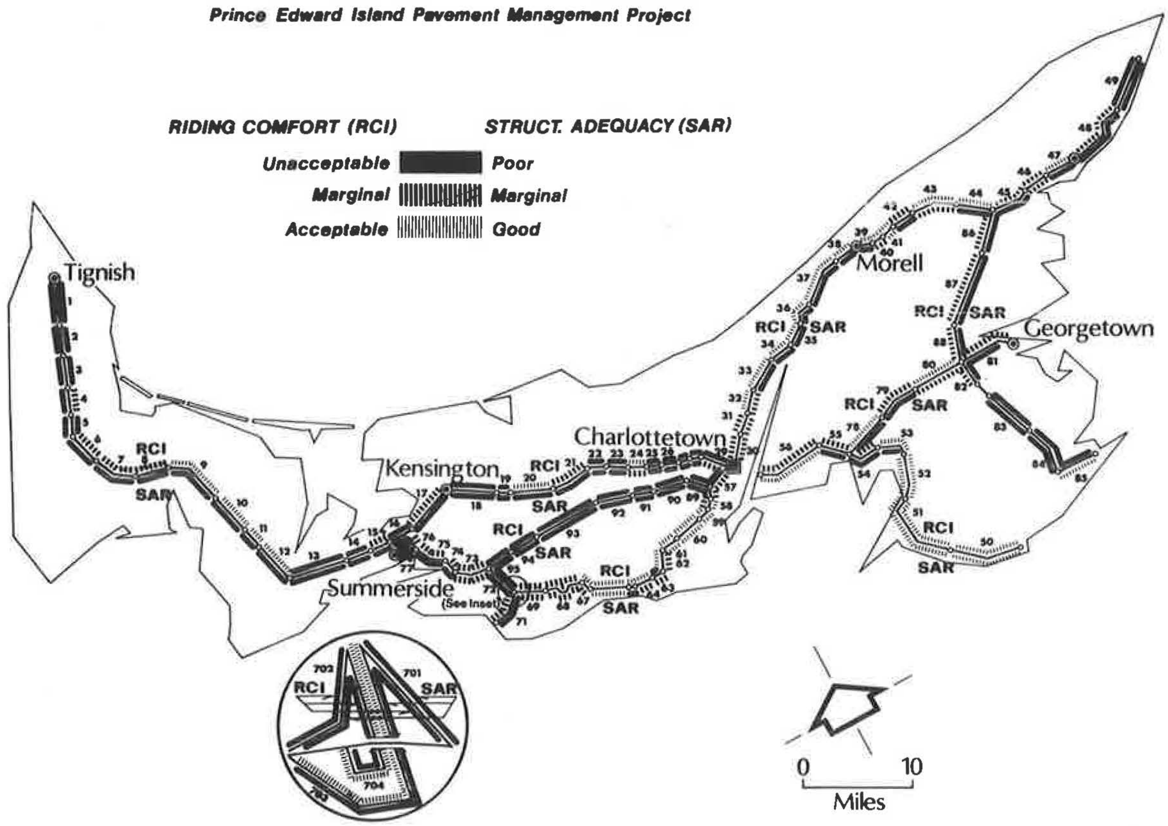
The remaining service life of each section is predicted by a performance-prediction submodel. This model, which is based on engineering experience, starts with the PSI of the section. Then, by using a technique known as Markov modeling, SI levels in future years are predicted. The remaining service life is then defined as the time required for a section to drop from its PSI to the minimum acceptable SI.

The sections that reach their minimum acceptable SI within the programming period of 10 years become candidate projects and are selected for further analyses. The five alternative rehabilitation strategies described previously are generated for each candidate project, and an economic analysis is conducted.

One of the most attractive features of the system is its capability for testing the effects of project timing. This means that a project does not have to be rehabilitated in the year in which it first reaches its minimum acceptable SI. It can be delayed (by using extensive routine maintenance) and may not be rehabilitated at all (in the programming period), depending on the economics involved and the budget available. Such factors as increased maintenance costs associated with the delay need to be taken into account in this case.

Similarly, a project can be rehabilitated before it reaches its minimum acceptable SI if adequate funds are available. The program allows the user to specify the number of acceleration years desired in the analysis (projects were allowed to accelerate a maximum of five years in this analysis). Each road section analyzed in the program is then accelerated by the specified number of years, and detailed analyses are conducted starting from that year if the serviceability level for that year is not greater than a previously set limit. An upper SI

Figure 2. Riding comfort and structural adequacy of Trans Canada and major-arterial sections.



level of 6.5 was used in this project.

Economic analyses are conducted for each implementation year and for each combination of project and rehabilitation strategy. The output of this phase is a list of sections (projects), alternatives and their direct costs to the agency, and user benefits (i.e., savings in vehicle operating costs due to improved pavement conditions) for each possible implementation year in the programming period of 10 years.

In the third phase, this information is used in a mathematical-optimization (linear-programming) model that establishes pavement-improvement priorities on the basis of benefit maximization and budget constraints. This model also recommends an optimum rehabilitation strategy for each project (section) considered in the analysis.

The final output of the system, therefore, is an optimum rehabilitation strategy and implementation year for each project. It is based on the maximization of user benefits and at the same time ensures that the agency will stay within its budget in each year throughout the programming period.

PRINCIPAL FINDINGS

Present Status of Network

In general, the Trans Canada sections analyzed in the project were found to be in a better condition than the major-arterial sections were. The average SAR and RCI levels calculated for these sections were significantly higher than the averages calculated for major arterials. The average SAR on Trans Canada was 5.2 as opposed to one of 2.1 on arterials. Similarly, the average RCI on Trans Canada was 5.9, whereas major arterials had an average RCI of 5.3.

The 1979 CI, RCI, SAR, and SI levels for both Trans Canada and major-arterial sections were presented in Table 1. The 1979 RCI and SAR levels are also presented in a coded form in Figure 2. This figure indicates that the Trans Canada sections are generally stronger (for the traffic volume that they carry) than are the major-arterial sections. They also provide a better ride for the user than do the major-arterial sections. As a result, the overall SIs of the Trans Canada sections are higher than they are for the major-arterial sections.

Of the 25 Trans Canada sections analyzed in the project, 11 were found to have PSIs of less than 5.0. Thus, they were candidates for immediate rehabilitation, since these levels are unacceptable.

Similarly, 57 of the 73 major-arterial sections had unacceptable SI levels (below 4.5). These were also candidates for rehabilitation in 1979.

The 11 Trans Canada and 57 major-arterial sections that had unacceptable PSI levels were identified as candidate projects for 1979. The other sections became candidate projects in the year in which they were expected to reach their minimum acceptable levels. In the following, the candidate projects for each year in the programming period of 10 years are described.

Determination of Need for Rehabilitation

The road sections that had PSI values that were greater than their minimum acceptable levels were analyzed in the program by using the Markov model to predict their future performance curves. A road section was identified as requiring a rehabilitation in the year in which it reached its minimum acceptable level (i.e., when the predicted performance curve crossed the minimum acceptable level).

The pavement-rehabilitation needs for Trans Canada sections are as follows: 1979, sections 54, 55, 57, 63, 64, 65, 67, and 701-704; 1981, sections 52, 53, 58, 62, 68, 69, and 71; 1983, sections 56, 59, and 61; and 1986, sections 60 and 66. The total is 23. This means that only two sections (50 and 51) had reasonably high SI levels and thus were not expected to reach their minimum acceptable levels within the 10-year programming period. They are not listed above and were excluded from the analyses.

All 73 major-arterial sections considered in the project were found to require rehabilitation within the programming period. Of this total, 57 sections were identified as requiring rehabilitation in 1979. The remaining 16 sections that appeared to have acceptable SI levels were expected to reach their minimum acceptable SI levels within the first five years of the programming period.

Pavement Improvement Priorities

The sections listed above would have been the priority list for Trans Canada sections if unlimited funds were available. Under budgetary constraints, however, some projects could not be built in the year in which they required major rehabilitation. They may have been delayed in time or not scheduled for rehabilitation at all, depending on the funds available.

Table 2 gives the recommended pavement-improvement priorities from application of the optimization model for the Trans Canada sections. Inspection of Table 2 indicates that only 6 of the 11 projects identified as candidate projects in 1979 could actually be built in that year. Hence, they were delayed and scheduled for subsequent years, depending on their economics. Section 57, for example, was delayed until 1980 and section 67 was delayed until 1982.

Also given in Table 2 is the type of rehabilitation applicable to each section scheduled for rehabilitation within the 10-year programming period. A 140-mm overlay (A3), for example, was recommended for section 56 in 1981. Similarly, reconstruction (A4) was recommended for section 63 in 1979.

The incremental solutions given in Table 2, which result from the budget constraints, may appear to be unrealistic. Of section 52, 57 percent, for example, was scheduled for reconstruction in 1983. This could be handled by reconstructing one-half of the section in 1983 and the other half in 1984, which could allow payment to be made from both the 1983 and the 1984 budgets. Another possibility is full reconstruction in 1983 with partial payment made in 1983 and the balance payable on January 2, 1984, or on the first day of the 1984 fiscal year. This could then be accounted for as a carry-over cost on the 1984 budget, a common practice in some areas of the country.

However, these approaches may be impractical in a contractual sense. Probably the best plan would be to award contracts on the other section (i.e., 71) first, compare the total bid price with the cost estimate, and then decide whether sufficient funds are still available to award a contract on section 52 or to delay it until 1984. If there were sufficient funds, another alternative would be to award a contract on one of the smaller jobs for 1984 (i.e., advance it to 1983).

This type of situation will always exist, since it is impossible to estimate costs and/or bid prices precisely. Also, there is always some error attached to estimating the future performance of any pavement and hence the action year in which rehabilitation is expected to be required. For these reasons, annual updating of both the actual costs and the network inventory and the priority analysis in turn is desirable. In other words, the priority list generated in any analysis is not necessarily the final solution, no matter how objectively it is based. Not only must it stand the test of reasonableness, per se, but the engineer, administrator, or politician must exercise the final responsibility for selecting the program of work and exercise the necessary judgment in making modifications where required (such as that discussed in the preceding paragraph).

The program scheduled all the 23 sections identified as candidate projects in the previous section. The periodic updating referred to in Figure 1 may, however, result in changes in the priority list of Table 2.

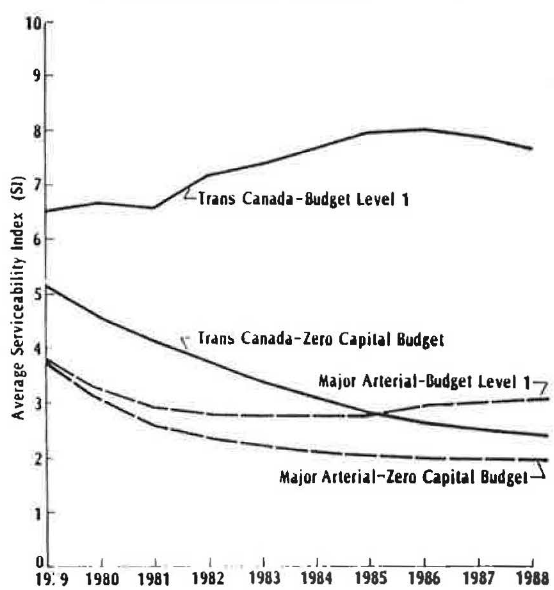
A similar priority list was developed for pavement improvements on the arterial sections. Fifty-nine of the major-arterial sections were not included on that priority list. This was because the budget available within the 10-year programming period was not enough to build all the major-arterial candidate projects analyzed. However, the same considerations regarding annual updating and exercising of judgment to make modifications where necessary, as previously discussed for the Trans Canada Highway sections, apply to these major-arterial sections. Of the 57 sections identified as requiring rehabilitation in 1979, only 6 were actually scheduled. The remainder of the projects were delayed or not scheduled at all, depending on their economics.

Budget-Level Analysis

The average SI levels for the Trans Canada and major-arterial networks in each year that would result from the implementation of the recommended priority lists are shown in Figure 3. The current mean serviceability level of the Trans Canada sections (about 5.2) was higher than that of the major-arterial sections (about 3.8), as mentioned earlier. The difference, however, increases significantly over the years as more and more Trans Canada sections are rehabilitated.

The implementation of the Trans Canada priority

Figure 3. Budget-level analysis for Trans Canada and major-arterial sections.



list in 1979, for example, would have increased the 1979 mean serviceability level to 6.5, whereas major-arterial sections would not have shown a significant increase in their 1979 mean serviceability level. The budget level used for Trans Canada sections (i.e., budget level 1 in Figure 3) resulted in a significant improvement in the mean serviceability levels over the 10-year programming period. The budget level used in the analysis for major-arterial sections (i.e., budget level 1) does not appear to have been sufficient to maintain the current status of the sections. Although there was an improvement compared with the case of zero capital budget (nothing is done except routine maintenance), the allocation of more funds to the major-arterial network appears to be justifiable.

Although the case of zero capital budget is an extreme one, it provides a good illustration of how a currently good road network could be allowed to deteriorate to an unacceptable level if funds for rehabilitation were cut off, which has in fact happened in some areas of the United States. An additional impact of such an action would be significantly higher user and maintenance costs and the likelihood of losing much of the existing investment (i.e., complete reconstruction would eventually be required).

SUMMARY AND IMPLICATIONS FOR THE FUTURE

The pavement management implementation project described in this paper was undertaken to provide PEI with

1. An objective data base for determining pavement improvement needs, and
2. An objective means for determining the most economical combination of projects, improvement strategies, and times of implementation for the portion of the road network considered.

The project involved a field inventory on some 500 km of PEI's Trans Canada and major-arterial networks, analyses of the inventory data to establish the improvement needs within a 10-year programming period, an economic analysis to determine the optimum list of pavement-improvement priorities, and an assessment of the effectiveness of the expected budget with regard to the average network serviceability.

The results of the project described in this paper have some major implications. The general one is the guidance provided for future management of the province's network of paved roads.

However, the results also indicate that if funds for rehabilitation of Trans Canada sections are decreased, the average serviceability level of these sections could decrease very substantially (Figure 3). Also, the arterial sections will further deteriorate below their current low level for the level of funding expected.

The investment the province has in its system of paved roads, the growing importance of this system, the increasing cost of maintaining it at a desirable service level, and its benefit to the province cannot be overemphasized. In our cost- and energy-conscious society, pavement management systems will continue to increase in importance to highway administrations at all levels of government. PEI is one of the first provinces to adopt such a complete system. Based on the success of this implementation project, the province is considering the expansion of this system to its entire network of paved roads.

The province has also recognized the importance in updating its inventory annually, since costs are constantly changing, some error in performance predictions is always likely, and the performance-prediction models can be readily defined as more field data become available. To this end, the province is attempting to expand its inventory equipment and field-testing capabilities so that by 1981 this annual inventory updating can be accomplished inhouse.

The project and its resulting implications illustrate how the concepts of pavement management developed over the past decade have come of age and are now being used to provide an objective and systematic means for planning and justifying pavement-network expenditures.

REFERENCES

1. Interim Report for the Pavement Management Implementation Project for the Province of Prince Edward Island. Pavement Management Systems, Ltd., Paris, Ontario, Nov. 1979.
2. D.A. Karan, D.H. Kobi, C.B. Bauman, and R.C.G. Haas. Subjective and Mechanical Estimates of Pavement Serviceability for Rural-Urban Road Networks. TRB, Transportation Research Record 715, 1980, pp. 31-36.
3. D. Kobi. ARAN Road Analyzer. Presented at the 60th Annual Meeting, TRB, 1981.
4. M.A. Karan. Alternative Means for Determining Structural Adequacy Rating of a Pavement in the Ontario Needs Study System. Regional Municipality of Waterloo, Waterloo, Ontario, Jan. 1978.
5. Flexible Pavement Serviceability (RCI)-Roughness Correlation for the Province of Prince Edward Island. Pavement Management Systems, Ltd., Paris, Ontario, Aug. 1979.
6. Pavement Management Guide. Road and Transportation Association of Canada, Ottawa, 1977.
7. M.A. Karan. Municipal Pavement Management System. Univ. of Waterloo, Waterloo, Ontario, Ph.D. thesis, May 1977.
8. M.A. Karan and R.C.G. Haas. Determining Investment Priorities for Urban Pavement Improvements. Proc., AAPT, Vol. 45, 1976.
9. M.A. Karan and R.C.G. Haas. Urban Pavement Management on a Coordinated Network: Project Basis. Proc., 4th International Conference on Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, 1977.

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Rehabilitation of Concrete Pavements by Using Portland Cement Concrete Overlays

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Overlays of portland cement concrete (PCC) are growing in popularity with paving engineers. This shift away from asphalt to concrete as an overlay material is due in part to some recent shortages in asphalt cement and the concomitant increase in cost of asphaltic concrete materials. Also, some engineers simply prefer concrete surfaces to asphalt for certain applications. PCC overlays are classified as bonded, partially bonded, or unbonded. Within these three classifications are continuously reinforced concrete, jointed concrete, and fibrous concrete overlays. Posttensioned prestressed slabs have also been used as overlays. Not all combinations of overlays and levels of bonding are compatible with all pavement types and all levels of distress. Thus each job must be evaluated as a separate project that uses the appropriate constraints. To evaluate the relative merits of the different types of overlays, a systematic approach to decision making must be used. The limitations and constraints of the different types of PCC overlays are discussed and a possible decision-criterion approach is described for use in evaluating the best overlay alternative.

Due in part to the increasing cost and in part to spot shortages of asphalt cement, pavement engineers are looking for alternatives to asphalt concrete for rehabilitation of portland cement concrete (PCC) pavements. One method being examined with increasing frequency is a PCC concrete overlay. A number of projects in recent years have demonstrated the economic and technical feasibility of this approach to PCC pavement rehabilitation (1-4). New equipment and technology developed in recent years have provided additional options for PCC overlay construction not available a decade ago, as discussed by Barenberg and Ratterree (5) and by Arntzen in a paper in this Record.

Concrete overlays can be defined and classified in several ways. Among the more obvious and popular classifications is one based on the degree of bond between the overlay and the existing slab, namely, bonded, partially bonded, or unbonded PCC overlays. Within each of these classifications, various types of PCC overlays might be considered, for example, continuously reinforced overlays, plain jointed overlays, reinforced jointed overlays, fibrous concrete overlays, and even prestressed (post-tensioned) concrete overlays. Not all types of PCC overlays are suitable for use with all types of existing concrete pavements. Furthermore, all types of PCC overlays may not be compatible for use with all levels of bond or all levels of distress. For best results, the type of PCC overlay must be matched to the existing pavement structure by type of slab, by condition, and by the degree of bond proposed.

Evaluating the true condition of the existing pavement is one of the most critical factors in selecting the best overlay option. This evaluation should reflect how the existing pavement will affect the behavior and performance of the overlaid pavement. Such an evaluation should be based on structural or behavioral considerations rather than on serviceability considerations.

Closely related to the pavement evaluation are the repairs and rehabilitation of the existing PCC pavements before overlaying. If most existing distress is eliminated prior to overlaying, then the effect of the existing pavement will be different than if the distress had been allowed to remain. Also, the method of repair is a significant factor in evaluating the pavement condition after repair.

Design procedures for PCC overlays have been developed over many years. Most of these procedures are empirical in nature and thus are valid only for the conditions for which they were developed. This leaves the problem of how to design PCC overlays for the new conditions, for which no experience is available.

Finally, after the pavement evaluation and the pavement repairs have been considered, the design procedures applied, and the final decision procedures and criteria implemented, the best PCC overlay must be selected and compared with alternative methods for rehabilitation of PCC pavements. This means careful matching of the advantages and disadvantages of each process and procedure with all others and then making engineering decisions based on facts rather than on personal opinions and biases. Clearly, too many overlay designs or other methods of rehabilitation are selected on the basis of what worked for other pavements rather than on careful selection of the best alternative for the particular job.

The basic concepts and steps outlined above will be expanded on in this paper, and recommendations and procedures for application will be described. Not all procedures described here were applied universally to all the rehabilitation projects described, but these guidelines were adhered to sufficiently to provide inputs for any future designs.

TYPES OF CONCRETE OVERLAY

Concrete overlays can be classified according to the level of bond developed between the overlay and the existing pavement slab. The three levels of bond generally recognized in this classification are fully bonded, partially bonded, and unbonded overlays. A summary of the three types of PCC overlays, the design procedures used, and the conditions and limitations for each is given in Figure 1 (6). A few comments on these conditions are appropriate here.

Bonded Concrete Overlay

Bonded concrete pavements are designed by simply determining the additional thickness of concrete needed to carry the anticipated traffic. This is expressed in Figure 1 as

$$T_r = T - T_o \tag{1}$$

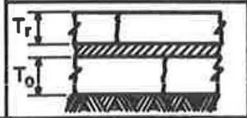
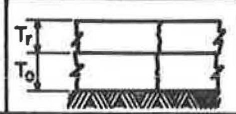
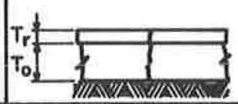
where

T_r = thickness of overlay required,
T = total thickness of PCC slab required for anticipated traffic and subgrade conditions, and
T_o = thickness of existing slab.

In determining the total thickness requirement T, the actual strength of the concrete in the existing slab must be used.

Since only sound existing pavements should be overlaid by using bonded concrete overlays, no condition-correction factor is used in this design.

Figure 1. Summary of PCC overlay designs for PCC pavements.

TYPE OF OVERLAY	UNBONDED OR SEPARATED OVERLAY	PARTIALLY BONDED OR DIRECT OVERLAY	BONDED OR MONOLITHIC OVERLAY
			
PROCEDURE	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL PLACE SEPARATION COURSE-PLACE OVERLAY CONCRETE.	CLEAN SURFACE DEBRIS AND EXCESS JOINT SEAL AND REMOVE EXCESSIVE OIL AND RUBBER - PLACE OVERLAY CONCRETE	SCARIFY ALL LOOSE CONCRETE, CLEAN JOINTS, CLEAN AND ACID ETCH SURFACE - PLACE BONDING GROUT AND OVERLAY CONCRETE.
MATCHING OF JOINTS IN OVERLAY & PAVEMENT: LOCATION	NOT NECESSARY	REQUIRED	REQUIRED
MATCHING OF JOINTS IN OVERLAY & PAVEMENT: TYPE	NOT NECESSARY	NOT NECESSARY	REQUIRED
REFLECTION OF UNDERLYING CRACKS TO BE EXPECTED	NOT NORMALLY	USUALLY	YES
REQUIREMENT FOR STEEL REINFORCEMENT	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT OR CONDITION OF EXISTING PAVEMENT.	REQUIREMENT IS INDEPENDENT OF THE STEEL IN EXISTING PAVEMENT. STEEL MAY BE USED TO CONTROL CRACKING WHICH MAY BE CAUSED BY LIMITED NON-STRUCTURAL DEFECTS IN PAVEMENT.	NORMALLY NOT USED IN THIN OVERLAYS. IN THICKER OVERLAY STEEL MAY BE USED TO SUPPLEMENT STEEL IN EXISTING PAVEMENT.
FORMULA FOR COMPUTING THICKNESS OF OVERLAY (T_r) NOTE: T IS THE THICKNESS OF MONOLITHIC PAVEMENT REQUIRED FOR THE DESIGN LOAD ON THE EXISTING SUPPORT C IS A STRUCTURAL CONDITION FACTOR T_R SHOULD BE BASED ON THE FLEXURAL STRENGTH OF	$T_r = \sqrt{T^2 - CT_o^2}$ OVERLAY CONCRETE	$T_r = \sqrt[1.4]{T^{1.4} - CT_o^{1.4}}$ OVERLAY CONCRETE	$T_r = T - T_o$ NOTE: THE ABILITY OF THE OVERLAID SLAB TO TRANSFER LOAD AT THE JOINTS SHOULD BE ASSESSED SEPARATELY EXISTING CONCRETE
MINIMUM THICKNESS	6"	5"	1"
APPLICABILITY OF VARIOUS OVERLAY TYPES — STRUCTURAL CONDITION OF EXISTING PAVEMENT — NO STRUCTURAL DEFECTS C = 1.0 *	YES	YES	YES
LIMITED STRUCT DEFECTS C = 0.75 *	YES	ONLY IF DEFECTS CAN BE REPAIRED	ONLY IF DEFECTS CAN BE REPAIRED
SEVERE STRUCT DEFECTS C = 0.35 *	YES	NO	NO
SURFACE CRACKS, SCALING, SPALLING AND SHRINKAGE CRACKS — NEGLIGIBLE	YES	YES	YES
LIMITED	YES	YES	YES
EXTENSIVE	YES	NO	YES

Bonded concrete overlays must be matched by type to the existing concrete slabs. That is, jointed concrete overlays must be used only on jointed concrete pavements, and continuously reinforced overlays can be used only on existing continuously reinforced concrete slabs. Furthermore, for bonded overlays the existing pavement slabs must be distress-free, since most distress in the existing slab will ultimately reflect through the overlay. Under very special conditions when thick (6 in or more) bonded overlays are used, bonded PCC overlays can be used over a cracked slab provided that the crack in the existing slab is tight and not working. Even then, reinforcing steel is recommended in the overlay across the crack. Obviously, when bonded overlays are used, the joints in the overlay must be matched to the joints in the existing pavement by both location and type.

One of the greatest advantages of the bonded concrete overlay is that a thin overlay can be used. Bonded concrete overlays as thin as 1 in have been used successfully on sound existing pavement, and bonded overlays 2-5 in thick are typical. Advantages of the thin overlays are lower costs and fewer problems in maintaining minimum overhead clearances and matching adjacent facilities. Because of the smaller amount of concrete used with the thin overlay, higher-quality concrete can be specified without significant adverse costs.

With respect to bonded concrete overlays, the proper preparation of the surface is most critical. Obviously, all dirt, grease, and unsound concrete must be removed before the overlay is placed. Some engineers feel that the old surface must be cleaned by removal of the existing surface by cold milling or similar procedures (1). There is evidence, however, that sandblasting and/or hydrobrooming by using water at high pressure (5000-7000 lbf/in^2) will adequately clean the surface unless grease and oil have penetrated deep into the existing concrete (1).

Neat cement or cement-sand grouts are used to promote bonding between the existing concrete and the overlay. No special additives are needed when these grouts are used, and they can be spread over the existing surface by using brooms (cement-sand or neat cement grouts) or by using pressure spraying (neat cement grout only). All grouts should be placed on a dry surface just before the fresh concrete is placed.

Partially Bonded Concrete Overlay

For partially bonded PCC overlays, the thickness design is based on the concept that the existing slab and the overlay act in part as a composite system; those portions of the pavement in which bond was achieved act essentially as a monolithic slab and partially support the portions of the slab that have little or no bond. The thickness requirements for partially bonded PCC overlays are determined from the following empirical relationship, shown in Figure 1:

$$T_r = (T^{1.4} - CT_o{}^{1.4})^{1/1.4} \qquad (2)$$

where T, T_r, and T_o are as defined for the bonded PCC overlays. The C-value is a condition index for the existing pavement, which can be defined as follows:

C = 1.0: no structural defects;
C = 0.75: limited structural defects; and
C = 0.35: severe structural defects.

As with the bonded concrete overlay design, the thickness must be determined for the anticipated traffic and support conditions by using the actual strength of the concrete in the existing pavement.

Partially bonded PCC overlays should also be used only on reasonably sound existing pavements, since most cracks in the existing slab will reflect through the overlay within a short period of time. Joints in the existing pavement must be matched by location in the partially bonded overlay.

Surface preparation of the existing concrete is much simpler than it is for fully bonded concrete. The only requirements for partially bonded overlays are that the surface be free of loose materials and that the existing concrete surface be sound. Ideally, the surface should be washed clean of all debris, and paint strips and heavy grease should be removed before overlaying. No grout or special additives are used to promote the bond when partially bonded overlays are used.

Since the partially bonded overlay and the existing pavement are not necessarily monolithic, minimum overlay thickness requirements must be rigidly adhered to. Ideally, the minimum thickness for partially bonded overlays is 6 in, although 5-in overlays have been used successfully. Unless joints are closely spaced, however, significant cracking between joints can be expected when thin partially bonded PCC overlays are used.

Unbonded Concrete Overlay

Unbonded overlays are intended to be used on existing pavements in which distress is too extensive and too severe to be effectively eliminated before overlaying. A separation course is used between the existing slab and the overlay to prevent the distress in the existing slab from reflecting through the overlay. A big advantage of this type of overlay is that it is not necessary to match the joints between the existing pavements and overlays or even to clean or seal these joints.

Fully unbonded PCC overlays behave eventually as slabs supported by a firm subgrade. Conceivably, one could therefore design an unbonded PCC overlay as a new slab by using the existing pavement only as support and assigning a k-value to the existing pavement. The problem here is one of determining the effective k for the existing pavement.

Unbonded PCC overlays are usually designed according to the following empirical relationship:

$$T_r = (T^2 - CT_o{}^2)^{1/2} \qquad (3)$$

where T_r, T, T_o, and C are as defined earlier for the fully bonded and partially bonded PCC overlays.

Although the design approach for unbonded concrete overlays is empirical, the basic idea is that the existing pavement serves as a support for the overlay. Consequently, all tipping or rocking slabs should be stabilized by slab jacking or sealed by using heavy rollers to provide a uniform support for the overlay.

The major disadvantage of unbonded PCC overlays is the greater thicknesses required and the concomitant higher costs and greater clearance problems. Minimum thickness for unbonded PCC overlays is 6 in and most overlays will probably be 7 or 8 in thick, depending on the traffic and the condition of the existing pavement.

EVALUATING EXISTING PAVEMENT CONDITIONS

One of the most confusing aspects of the design of PCC overlays is the problem of evaluating the condition of the existing pavement. According to the design equations shown in Figure 1, the condition of the existing pavement is expressed by a structural condition factor C. This factor varies from 1.0 for an existing pavement in near-perfect condition to 0.35 for a pavement that has a number of shattered slabs. The problem is that this structural condition factor is highly subjective, and only the values of 1.0, 0.75, and 0.35 are used for evaluation of existing pavements.

To determine when major rehabilitation is needed on concrete pavements, some form of serviceability rating system is frequently used (7). This approach to pavement evaluation does not provide the necessary structural information needed to design overlays. In recent years, a pavement condition index has been developed that will provide significant information on the structural health of the pavement (8). This type of information is of great value if rational overlay design processes are to evolve.

Use of nondestructive-testing equipment for evaluating PCC pavements has been suggested (9,10), but these procedures have not been fully developed or effectively implemented. The problem with nondestructive-testing evaluation of PCC pavements is that some pavements that show severe distress, such as D-cracking, due to environmental factors will in fact show excellent results under nondestructive-test loading. Experience shows, however, that such pavements are not good candidates for rehabilitation by using overlays, especially when the existing pavement is expected to carry a significant portion of the load.

None of the evaluation procedures currently in use deal directly with the most serious problems in PCC pavements, namely, the joints. After discussions of the problem of PCC pavement rehabilitation with a number of highway engineers, it became obvious that there was no viable and consistent procedure other than the visual one for determining which joints should be replaced before overlaying, which should merely be resealed, and which should be left untouched. If effective overlay or rehabilitation procedures are to be developed, procedures of evaluating the effectiveness and life of the joints in the existing pavements must be developed.

NEW APPROACHES TO PCC OVERLAY

In recent years, development of new technology and new equipment has provided the means for new approaches to be used with PCC overlays. Specifically, fibrous concrete has been used as an overlay in several locations (7,8,12). Some experiences with fibrous concrete have been good and others have not. The greatest advantage to the use of fibrous concrete rather than plain concrete would be in the reduction of the number of joints needed and elimination of the need to match joints carefully by both location and type between the existing overlay slabs.

Development of cold-milling equipment, which permits the removal of thin strips of the existing PCC surface, has spurred renewed interest in fully bonded PCC overlays (1,5). Cold milling eliminates the need for acid etching and provides a highly reliable bond between the existing PCC slab and the PCC overlay. There are also indications that sandblasting or combined sandblasting and high-pressure water blasting or similar types of surface preparation will result in an equally reliable bond between the two PCC layers without the problem of surface damage to the existing pavement slabs sometimes observed with the cold-milling operations (12).

One of the major problems with fully bonded PCC overlays is reflective cracking. If use of fibrous concrete could eliminate or even greatly reduce the reflective-cracking problem, then this procedure would appear to have great promise as a PCC overlay option. It has been used with only one pavement (Reno Airport), to my knowledge, and has performed well. Additional techniques must still be worked out, however, for how best to handle the joints or cracks in the existing pavement. With the reduction in reflective cracking found by using fibrous concrete, these problems may not be so severe as they are with other types of PCC overlays.

Another example of new developments in PCC overlays is the recent construction of a posttensioned PCC overlay at Chicago O'Hare International Airport, as discussed by Arntzen in a paper in this Record. Several airports in Europe have had excellent experience with newly constructed posttensioned pavements in which the posttensioned slab is placed on a stabilized subbase. The O'Hare project is the first example of the use of a posttensioned slab as an overlay. If the European experience with posttensioned pavements is positive, this could be a viable alternative as a low-maintenance overlay for premium pavements. Cost of this type of construction for rehabilitation would likely preclude the use of posttensioned overlays for any pavements except those on which heavy traffic would justify such cost because of high user cost for down time during maintenance operations. Costs per square yard for the posttensioned overlays are comparable with costs for a new PCC pavement for the same conditions.

In addition to the new techniques for PCC overlays, there are also advances in the technology for repair and rehabilitation of the existing pavements before overlaying. Principal among these developments are the partial-depth patching at joints by cold milling to sound concrete and placing a fully bonded partial-depth PCC patch and new methods for reinstalling effective load transfer across joints and cracks. Lift-out, lift-in procedures for slab replacement have been used in areas of heavy traffic that have high user cost for down time for pavements that have a high volume of traffic (13). Load-transfer devices to reestablish load transfer across existing cracks and joints or to tie the precast slabs to the old pavement have also been developed and are being evaluated (14,15). Leveling of faulted slabs by slab jacking and use of cold-milling equipment is not necessarily new, but this technique is being used with increasing frequency, according to several highway engineers consulted recently.

These are but a few of the new concepts and procedures being used in the rehabilitation of PCC pavements. No doubt other procedures could also be found. The point is that there is much room for ingenuity and engineering innovation in the area of pavement repair and rehabilitation. As more of our high-volume PCC pavements experience distress and with the increasing cost of conventional methods of rehabilitation by using asphalt concrete, it is likely that more innovations will be developed. The engineer should be aware of such developments and use them when these newer approaches can make PCC pavement rehabilitation more effective.

DECISION CRITERIA

Too frequently, the decision as to which type of rehabilitation to use is based on the least initial cost. Since there is an increasing number of heavy-volume PCC pavements that need rehabilitation and increasing pressure for getting the most for our rehabilitation dollar, it is necessary to develop added criteria and procedures for selecting the best overlay and rehabilitation scheme.

There are a number of factors that can affect the decision as to which pavement rehabilitation technique is best suited for any given pavement. These factors vary for different pavement and traffic conditions and for different levels of distress. Factors that might be considered in such an evaluation process include initial cost, average annual cost, design reliability, future traffic disruption and maintenance efforts, construction duration, energy consumption, and others.

Some of the factors that affect the design decisions are subjective and difficult to quantify. To relate the suitability of each alternative for a particular project, a ranking system can be developed similar to that outlined in the Federal Highway Administration's Value Engineering for Highways (16). In this approach, each evaluation or decision factor is assigned a value from 0 to 100 to reflect its relative importance in the final decision. Some factors and their relative importance are listed below. It is important to note that the relative importance value (RIV) for each factor may change from project to project.

	RIV	
Factor	Project 1	Project 2
Initial cost	25	20
Avg annual cost	20	20
Design reliability	20	20
Construction duration	15	20
Pavement manageability	10	5
Energy consumption	5	0
User inconvenience during construction	5	15

Table 1 shows how the various alternatives can be ranked by using this system. For each alternative, each factor is assigned a rating based on this factor's standing among all alternatives. For example, for initial cost, the alternative that has the highest initial cost will be assigned a zero rating and the alternative that has the lowest initial cost will have a rating of 100. These ratings are then multiplied by the RIV for each factor. By summing the products of the RIV and the rating value for all factors for each alternative, the numerical ranking of each alternative is deter-

Table 1. Calculation of ranking for three alternatives.

Factor	RIV	Project A Amount	Project A Rating	Project A Ranking	Project B Amount	Project B Rating	Project B Ranking	Project C Amount	Project C Rating	Project C Ranking
Initial cost ($)	25	10 437	0	0	8 709	52	13	7 159	100	25
Avg annual cost ($)	20	510 000	40	8	442 150	90	18	427 950	100	20
Design reliability	20	–	80	16	–	70	14	–	30	6
Construction duration (days)	15	–	100	15	–	100	15	–	20	3
Pavement manageability	10	–	90	9	–	50	5	–	20	2
Energy consumption (billion Btu)	5	204	0	0	89	75	2.5	50	100	5
User inconvenience	5	–	20	1	–	70	3.5	–	90	4.5
				49			71.0[a]			65.5

[a]Best alternative.

mined. The alternative that has the highest summation will be the most desirable alternative.

Obviously, the above procedure is not a precise calculation, but it is interesting to note that when engineers apply this technique, they frequently come up with the same final answer even though there was no coordination in the RIV or the rating values assigned. Furthermore, it is not unusual that the best alternative arrived at in this manner is not the same as the alternative the engineer would have chosen without the evaluation. But when asked how the evaluation should be changed, no one has any suggestions and all usually agree that the alternative indicated by the procedure is probably the best one.

Perhaps the best feature of this approach to decision making is that it forces the engineer to consider all factors involved in these decisions in a rational manner. If a systematic procedure is not used, some factors are often forced into the background and not properly taken into account in the final decision.

SUMMARY

The use of PCC overlays is a viable method for rehabilitating existing PCC pavements. However, there are a number of types of PCC overlays (bonded, partially bonded, and unbonded overlays) that may be subdivided as to type of pavement. Not all types of PCC overlays are suitable for use with all types of existing PCC pavements. Also, the level and density of distress in the existing pavement may severely limit the options available to the designer.

Because of the number of options available, all PCC rehabilitation plans should start by making a careful evaluation of the existing pavement. Such evaluation should include the support conditions of the existing pavement and the structural condition of the existing slabs. When the existing conditions of the slab are evaluated, particular attention must be given to the condition of the joints, especially to their load-transfer efficiency. Parts of the above evaluations can be made with nondestructive-testing equipment and thorough visual inspections of the pavements by trained observers.

Finally, after all data on the existing conditions have been gathered, the designer should make the necessary decisions as to which of the overlay options are or are not valid because of existing pavement conditions. Such decisions must by necessity include the two options of repairing the distress in the existing pavements first or of not repairing it. With these two options, alternative rehabilitation programs should be developed that consider all valid overlay design approaches.

To ensure a careful consideration of all factors, a systematic approach to the evaluation of all overlays should be taken. The approach shown here is one scheme that can be used. Other schemes may also be effective. The important point is to use some logical scheme in the decision-making process so that all factors are properly considered.

Finally, the design engineer should be alert for any and all improved procedures for rehabilitating PCC pavement. All pavement rehabilitation requires innovative engineering.

REFERENCES

1. M.I. Darter and E.J. Barenberg. Bonded Concrete Overlays, Construction, and Performance. U.S. Army Engineer Waterway Experiment Station, Vicksburg, MS, 1980.
2. Design and Construction--Continuously Reinforced Concrete Overlays. Continuously Reinforced Pavement Group, Concrete Reinforcing Steel Institute, Chicago, IL, 1973.
3. AASHTO Interim Guide for Design of Pavement Structures, 1972. AASHTO, Washington, DC, 1974.
4. ACI Committee 544. State of the Art on Fibrous Reinforced Concrete. Journal of American Concrete Institute, Proc. Vol. 70, No. 11, 1973.
5. E.J. Barenberg and B.L. Ratterree. Fully Bonded Concrete Overlay for an Airport Runway. Proc., Air Transport Division of ASCE, International Air Transportation Conference, New Orleans, LA, Vol. 2, May 1979, p. 383.
6. G.K. Ray. Design of Concrete Overlays for Pavements. Journal of American Concrete Institute, Aug. 1967.
7. W.A. Yrjanson. A Review of Field Applications of Fibrous Concrete. TRB, Special Rept. 148, 1974, pp. 69-79.
8. F. Parker, Jr., and J.L. Rice. Steel Fibrous Concrete for Airport Pavements. Proc., International Conference on Concrete Pavement Design, Purdue Univ., West Lafayette, IN, 1977.
9. Use of Nondestructive Testing Devices in the Evaluation of Airport Pavements. Federal Aviation Administration, FAA Advisory Circular AC 150/5370-11, June 4, 1976 with change 1, July 11, 1980.
10. Crawford, Murphy, and Tilly, Inc. Summary of Conclusions of Design Study; East-West Tollway, Maperville Road to Aurora Plaza. Illinois State Toll Highway Authority, Oak Brook, 1980.
11. W.L. Grambling and T.H. Nichols. Steel-Fiber-Reinforced Concrete. TRB, Special Rept. 148, 1974, pp. 160-165.
12. M.J. Knutson. Evaluation of Bonded, Thin-Life, Non-Reinforced Portland Cement Concrete Resurfacing and Patching. Presented at 62nd Annual Meeting, AASHTO, Birmingham, AL, 1976.
13. J.E. Simonsen. Development of Procedures for Replacing Joints in Concrete Pavements. Michigan State Highway Commission, Lansing, Res. Rept. R-968, 1975. NTIS: PB 254797/4ST.
14. W.B. Ledbetter, R.L. Lytton, S.C. Britton, W.G. Sarver, H.L. Furr, J.A. Epps, J.B. Mahoney,

and N.F. Rhodes. Techniques for Rehabilitating Pavements Without Overlays--A Systems Analysis. Volume 1: Analyses. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-78-108, 1978.
15. R.W. Grau. Strengthening of Keyed Longitudinal Construction Joints in Rigid Pavements. U.S. Army Engineer Waterway Experiment Station, Vicksburg, MS, Final Rept. FAA 12-D-72-106, 1972.
16. Olympic Engineering Corporation. Value Engineering for Highways. FHWA, U.S. Department of Transportation, 1979.

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Pavement Management Study: Illinois Tollway Pavement Overlays

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Since 1967, when Byrd, Tallamy, MacDonald and Lewis (BTML) performed the original pavement maintenance study for the Illinois State Toll Highway Authority, there have been major changes in the characteristics of the highway, volume of traffic, and in the pavement composition itself. Several studies have provided information to update the original maintenance and rehabilitation program, and the study reported here has created a continuity in this process. As the result of the comprehensive pavement evaluation by BTML, data have been accumulated on current conditions of serviceability, slipperiness, surface defects, and deflection. These factors were considered individually as well as collectively to provide recommendations for improvements or rehabilitation. Current pavement condition was determined through visual and instrument surveys to provide present-serviceability-index factors and computations, traffic and axle-load analyses, and skid numbers for each of the three tollways in each direction. The visual pavement deficiencies—cracking, patching, faulting, and pumping—on rigid pavements were addressed by the visual survey. The instrument survey was concerned with the determination of roughness, skidding, and deflection data. Pavement condition was determined through the study of traffic volume, lane distribution, axle load, and the number of axle repetitions. Cumulative 18-kip single-axle loads were determined for the tollway. An integral part of a pavement management system is an adequate data base. The evaluation performed by BTML compiles the data necessary to create a format adaptable for use in an effective pavement management system for the tollway. The pavement management framework is a management tool to aid consistency and optimization in the decision process. It is designed to expand decision-making capability as well as to provide necessary feedback on these decisions.

The Illinois Tollway consists of three toll highways--the Tri-State, East-West, and Northwest Tollways, as illustrated in Figure 1. Together they total approximately 243 centerline miles, of which 104 miles have three traffic lanes in each direction. In addition to the main-line mileage, the Illinois Tollway consists of several access ramps, interchanges, and toll-collection facilities. The Illinois Tollway is a high-level system that serves motorists in the metropolitan Chicago area as well as throughout the state of Illinois. Segments of the tollway system serve more than 100 000 vehicles daily, which includes Interstate transport and localized commuter travel.

Management decisions are made as a part of normal daily operations for an active highway system such as the Illinois Tollway. The pavement evaluation and rehabilitation criteria provided as a part of this study are intended as management tools to aid the decision maker. They are designed to improve the efficiency and consistency of the decision-making process.

Current pavement-rehabilitation needs are in part a function of management decisions made in the past. Likewise, decisions made today will have an impact on future pavement-rehabilitation needs and, consequently, costs.

HISTORY OF PAVEMENT EVALUATION

The American Association of State Highway Officials (AASHO) Road Test conducted near Ottawa, Illinois, during 1956-1961 produced basic concepts about the evaluation of existing pavement conditions and the relationship of a pavement service life to the number of axle loads to which it is subjected.

Realizing the importance of this approach to the prediction of pavement service life, tollway officials in 1967 engaged Byrd, Tallamy, MacDonald and Lewis (BTML) (then Bertram D. Tallamy and Associates) to evaluate long-range pavement-maintenance needs for the tollway system. As part of that study, a detailed pavement condition survey was conducted on the entire tollway system. One of the principal purposes of the field inspection was to obtain the factual data required for the serviceability-index equation developed at the test road.

In the application of road-test equations to the tollway, it was necessary during 1967 to undertake a comprehensive study of traffic. This was required to estimate the characteristics and amount of traffic that had used the road between each interchange section since it had been opened. Similarly, extensive axle-load computations were made to determine the number and magnitude of axle loads. Pavement design and present-serviceability-index (PSI) values were then used to plot the service-life curves for each average PSI section between major interchanges. The year at which the pavement is estimated to reach a terminal serviceability condition (TSI) was determined from these curves.

The need existed to extend and make slight modifications to the service-life curves developed during the 1967 study because of the environmental and traffic effects on the tollway pavements. Also, there was a need to develop new service-life curves for those pavement sections that had been resurfaced since 1967. The tollway engaged BTML to perform the necessary observations, measurements, calculations, and analyses to adjust these curves and extend the resurfacing schedule in 1969, 1971, and 1975.

More than 20 years have passed since the original pavement evaluation, and major changes have occurred in both the composition and the volume of traffic on the tollway, as well as in the pavement structure itself. For a comprehensive pavement evaluation, skid data and structural-strength data are obtained in addition to serviceability indices and a visual survey in 1979.

Figure 2 illustrates the study approach for the systemwide pavement evaluation and project identification for the Illinois Tollway.

PAVEMENT-CONDITION ANALYSIS

The pavement-condition analysis survey is a three-part procedure. The first two parts consist of collecting field data through visual and instrument surveys. The third part reduces the field data to numerical values of PSI, skid numbers (SNs), and structural deflections.

By using previous studies as a data base, service-life curves can be effectively and adequately updated by selecting representative sampling areas for which detailed pavement-condition surveys would be performed.

The current study recognizes the need to base pavement-rehabilitation decisions on safety and structural capacity in addition to adequate serviceability. This pavement evaluation includes collection of field data necessary to calculate PSIs, time to reach terminal serviceability, SNs, and structural indices. All of these form a basic data base for development of a pavement-inventory system.

VISUAL SURVEY

In order to conduct a detailed visual survey representative of the tollway pavement sections, analyses of roughness measurements, rut-depth measurements, structural-deflection measurements, average daily

Figure 1. Illinois Tollway.

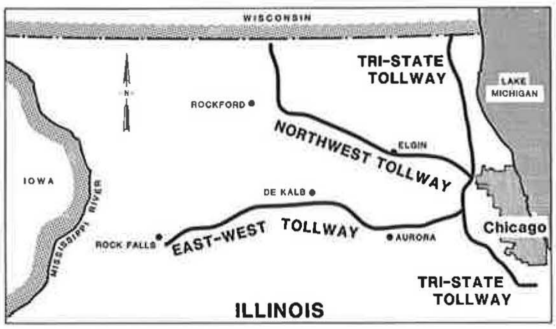


Figure 2. System pavement evaluation and project selection.

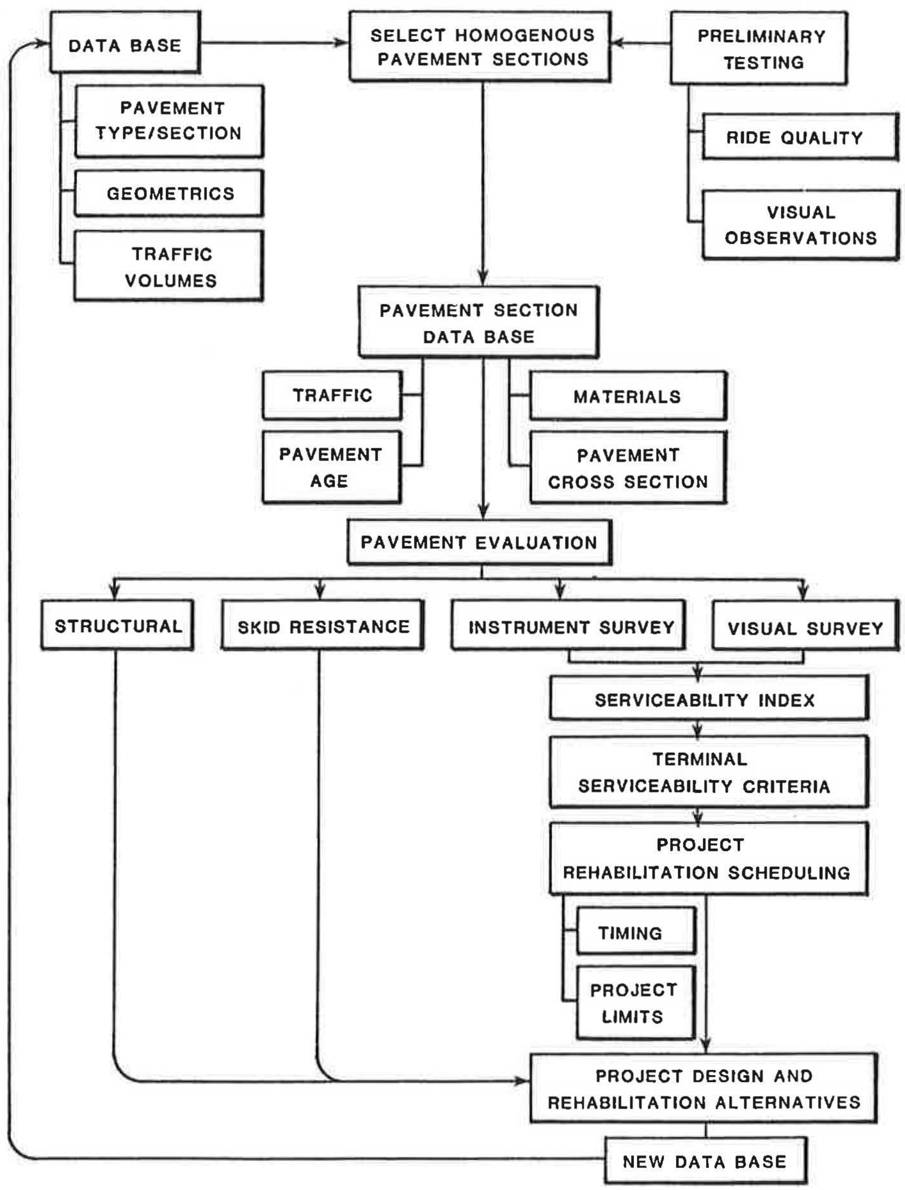


Figure 3. Lane numbers.

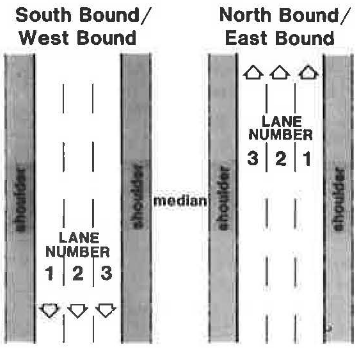
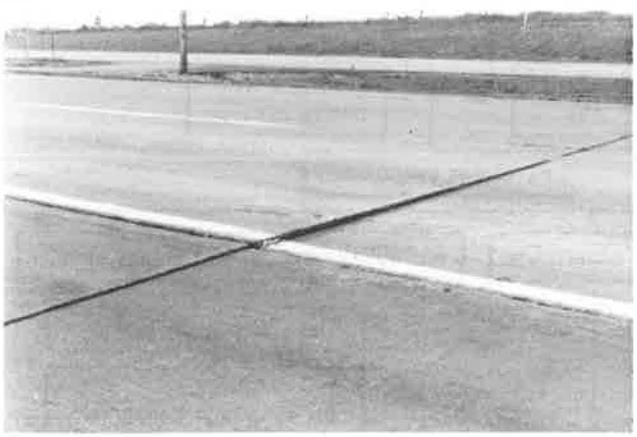


Table 1. Visual-inspection limits and corresponding pavement sections.

Tollway	Visual-Inspection Limits (milepost)	Detailed Visual Survey Limits	
		Milepost	Direction
Tri-State	0-16	14.0-14.5	NBL and SBL
	16-30	26.5-27.0	NBL and SBL
	30-42	36.5-37.0	NBL and SBL
	42-70	54.5-55.0	NBL and SBL
	70-77	76.0-76.5	NBL and SBL
Northwest	0-5	3.5-4.0	EBL and WBL
	5-17	13.0-13.5	EBL and WBL
	17-24	22.5-23.0	EBL and WBL
	24-63	43.0-43.5	EBL and WBL
	63-76	64.5-65.0	EBL and WBL
East-West	60-68	66.0-66.5	EBL and WBL
	68-96	82.5-83.0	EBL and WBL
	96-129	109.0-109.5	EBL and WBL
	129-133	130.5-131.0	EBL and WBL
	133-144	139.5-140.0	EBL and WBL
	144-156	151.0-151.5	EBL and WBL

Note: NBL = northbound lane; SBL = southbound lane; EBL = eastbound lane; WBL = westbound lane.

Figure 4. Pavement faulting on East-West Tollway, westbound lanes, milepost 109.5.



traffic volumes, and pavement compositions were used. Based on these analyses and a visual inspection of the entire tollway system for pavement defects, limits were established for a detailed visual survey. Results of cracking and patching counts within these limits were used in PSI determinations for all pavement sections contained within these limits.

Although all pavement lanes were observed and lane conditions recorded in the representative sections (Figure 3), the formula data were developed only from those conditions recorded in the outside driving lane (lane 1). The outside driving lane generally represents the most critical condition and is the controlling lane when resurfacing or rehabilitation measures are scheduled. Visual-inspection sections and the inspection limits representative of them are summarized in Table 1.

Deficiencies common to rigid pavements were inspected and recorded. To provide data for the PSI computations for rigid pavements, the visual-survey team observed and recorded two specific types of pavement distress--cracking and patching. These measurements were in accordance with criteria used at the AASHO Road Test.

In addition to patching and cracking data obtained for use in the serviceability computations, the visual-survey team inspected the substantial cracks, joints, and edges of pavement to determine the extent of faulting, spalling, and pumping. These supplemental data provide valuable information for immediate maintenance and rehabilitation program planning. In instances in which low PSI values are found, the type of distress may suggest specific causes and the corrective action required.

Faulting

Faulting is defined as a vertical displacement of the pavement slabs adjacent to a joint or crack (Figure 4). In the case of longitudinal joint faulting, settlement is generally confined to the lane that receives the heavier traffic. When transverse joint or crack faulting is present, the impact from axle loads generally causes settlement of the downstream slab. Faulting was measured to the nearest 0.125 in. Faulted transverse cracks and joints were recorded by numerical count.

Pumping

Pumping is the term used to identify the ejection of water and/or subbase material along the pavement joints, cracks, and edges caused by movement of the pavement slab that results from the passage of heavy axle loads. It can be detected by the stained appearance of the pavement surface adjacent to the joint, crack, or edge and/or by deposit of fine material adjacent to the pavement.

The pavement surface was inspected for other forms of deterioration, which included blowups, corrugations, disintegration, frost heave, pitting, popouts, settlements, curling, and warping.

INSTRUMENT SURVEYS

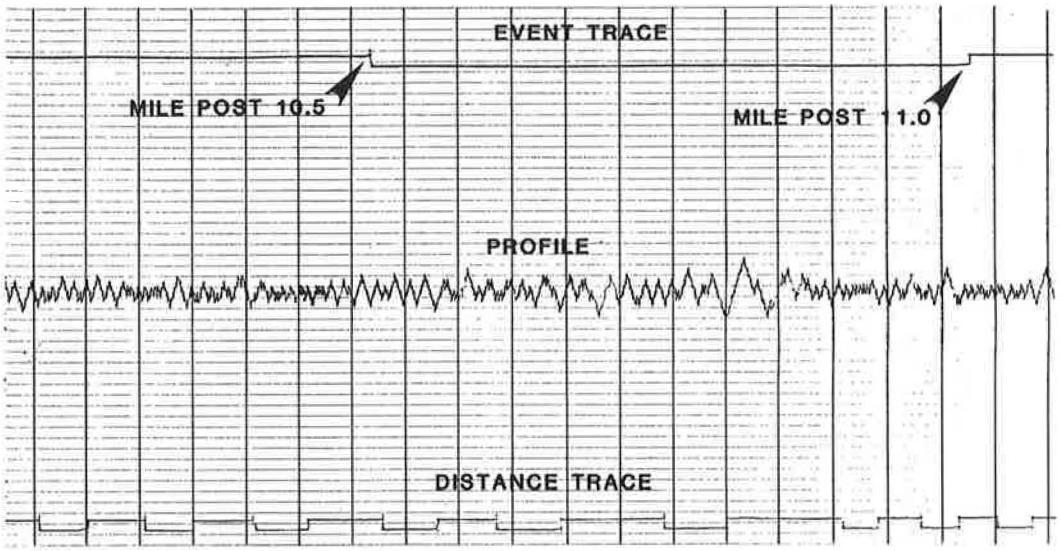
Instrument surveys were conducted to measure road roughness through the use of the Mays ride meter. SNs were determined by using the Law skid trailer and deflection by using the Dresser Atlas Dynaflect.

Roughness

Although the visual survey obtains data that reflect the deterioration of the pavement surface, the roughness survey records roadway ride-quality characteristics. Ride-quality deterioration is known to result from the cumulative effects of the pavement environment, mainly traffic loadings and climate. This phase of the field survey consisted of taking measurements by using a Mays ride meter to obtain variances from the longitudinal profile of the pavement surface. The Mays ride meter was mounted in a 1979 Chevrolet Impala to obtain roughness data.

Roughness surveys conducted in 1967, 1969, 1971, and 1975 used the Bureau of Public Roads (BPR) roughometer to determine roughness. Consequently, it was necessary to calibrate the Mays ride meter for use in serviceability equations that had a

Figure 5. Typical Mays ride meter roughness profile.



common base. Correlations were made by using the Illinois Department of Transportation BPR-type roughometer, which yielded a correlation coefficient of 0.91 from an analysis of variance.

Roughness measurements were made for each mile of the tollway in lanes 1, 2, and 3 in both directions. Roughness measurements were also made for each ramp on the tollway system. Figure 5 illustrates a typical roughness profile.

Roughness data used in PSI determinations represent an average roughness (in inches per mile in lane 1) of all miles in that particular pavement section. Lane 1 is generally the decision lane that controls pavement overlay or rehabilitation.

On the main line, a constant test speed of 50 mph was maintained. The beginning and ending of bridge structures were noted on the profile-event trace and were factored out from the pavement sections. Inches of roughness per lane mile of pavement were then determined.

Interchange ramps were surveyed at their posted speed. When two lane ramps were encountered, only lane 1 was tested. Ramp lengths were recorded and the corresponding inches of roughness per mile determined.

Skid

Pavement skid testing was conducted to obtain an initial skid-resistance survey of the tollway system. SNs provide a friction value for the tire-pavement interface representative of potential wet-weather skidding.

Skid testing was conducted by using a Law model 965 skid-measurement system owned by the Transportation Research Center of Ohio. The skid trailer nozzle is an Ohio State University nozzle that provides a uniform trace width of approximately 7 in in front of the test tire.

These tests were conducted in conformance with ASTM E274. Tests were conducted on a 500-ft section that began at every 0.50-mile post marker in lane 1 for each direction of travel on the main line. Tests were conducted at the standard 40 mph.

To obtain SNs at the main-line toll plazas, a series of tests was conducted at the approach, and a series of tests was conducted leaving the lane-2 toll gate. At selected interchange ramps and loops, tests were conducted at the standard 40 mph or the highest speed below this value that could be achieved depending on roadway alignment and traffic conditions. When possible, three readings were obtained per ramp.

Deflection

Structural capacity can be evaluated indirectly by noting the defects in an existing pavement or by measuring the deflection of the pavement system under an applied load. Typically, pavement deflections are measured at specified or selected locations, usually at critical sections detected by other routine monitoring. In most cases, deflection measurements are used for the design of sections that are candidates for rehabilitation. To aid in establishing a data base for a tollway inventory system, representative deflection data indicative of structural capacity and joint efficiency were obtained on the main-line pavements. A Dresser Atlas Dynaflect was used for the deflection survey.

Deflection measurements were obtained in lane 1 at the center of concrete slabs and across pavement joints. Testing frequency routinely varied from 0.25 to 0.50 mile. In sections where deflection data produced very little variance, testing frequency was increased to as much as 1-3 miles. At 6 percent of the testing locations, two test replications were made to ensure instrument and operator repeatability. On the tollway's main line, 314 test locations were evaluated.

PRESENT SERVICEABILITY INDEX

To permit orderly processing of field data and computation of PSI values, field records for each type of pavement distress were assembled. Visual and roughness data were tabulated for each recording segment. The data were then processed to develop the cracking, patching, and roughness factors for the PSI equation.

As part of the National Cooperative Highway Research Program, a study was conducted to develop PSI equations by using various models of roughometers and profilometers (1). A study conducted by Purdue University led to the development of a modified AASHO Road Test equation for obtaining the PSI value of a rigid pavement when the CHLOE profilometer is used. This equation produces PSI values that differ only slightly from those obtained by using the original AASHO Road Test equation. The Purdue equation was modified by the Illinois Department of Transportation to allow use of the Illinois BPR-type roughometer to measure roughness.

Both these equations give essentially the same results. Since serviceability data collected on the tollway system previously were compiled by using the modified AASHO equation, it was decided to continue the use of that equation adjusted for Mays ride meter roughness measurements. Thus, for rigid pavements, Equation 1 is used for PSI computations:

$$PSI = 12.00 - 4.27 \log RI - 0.09\sqrt{C+P} \quad (1)$$

$$RI = 63.74 + 0.29MRM \quad (2)$$

where

MRM = roughness values from the Mays ride meter,
RI = roughness index,
C = cracking factor, and
P = patching factor.

TRAFFIC AND AXLE-LOAD ANALYSES

Traffic data are required to determine the axle loadings that have occurred on the pavement sections of the tollway and to allow the prediction of future traffic loadings to which these pavement sections will be subjected. The data collected include traffic volume, vehicle classification, lane distribution, and variations in axle loads for each vehicle classification.

Traffic Volume

Traffic volumes were obtained directly from the Illinois Tollway's annual traffic reports. These reports were compiled by the Traffic Division of the tollway's Engineering Department.

The traffic data are summarized from traffic volumes at the main-line toll plazas for the years 1960, 1965, 1970, 1975, and 1978. All traffic-volume data obtained from these annual traffic reports were summarized in average daily traffic (ADT) figures.

When axle loads on pavement sections are estimated, commercial vehicles have a far more detrimental effect on pavement life than does noncommercial traffic. For this reason, a separate analysis was made of commercial ADT for the years 1960, 1971, 1975, and 1978. It was found that commercial ADT volumes did not follow total volume trends, particularly for combination-type vehicles. Therefore, it was not appropriate to express commercial traffic as a simple percentage of total traffic. Total commercial traffic volume had to be grouped by single-unit and combination vehicles. Single-unit commercial vehicles were found to be a simple percentage of total volume and, in some instances when only total commercial units were reported, provided the basis for a split between single-vehicle and combination-vehicle traffic.

Vehicle Classification

For the purpose of assessing tolls, the Illinois Tollway has grouped all vehicles into nine classes. However, it was necessary to combine these nine vehicle classes into the six axle classification categories defined at the AASHO Road Test to determine equivalent 18-kip single-axle loadings. The tollway classes and the corresponding AASHO axle groupings and vehicle classifications that BTML used to analyze axle loadings are given below (SV = single vehicle; CV = combination vehicle):

Tollway Class	AASHO Description	AASHO Class
1,7,8	Four-tire SV	1
2	Two-axle/six-tire SV	2
3	Three-axle SV	3
3	Three-axle CV	4
4	Four-axle CV	5
5,6	Five-axle or more CV	6

As determined in the 1968 long-range pavement maintenance program report prepared by BTML, tollway class-7 and class-8 vehicles were grouped into the class-1 axle category because they correspond to AASHO class-1 vehicles except that they are towing a one-axle trailer (class 7) or a two-axle trailer (class 8). The tollway class-3 vehicles included both single and combination three-axle vehicles.

By using the tollway to determine the mix of vehicle classifications, statewide average vehicle classifications were determined from the Illinois Department of Transportation W-4 loadometer tables. By applying these percentages to actual traffic volumes for each pavement section on the tollway, the number and mix of commercial vehicles in each vehicle classification were determined.

Lane Distribution

The distribution of traffic volume by lane was determined directly from field data. Since the commercial vehicles are important in determining axle-load applications, only vehicles in classes 2 through 6 were examined for lane-distribution purposes. For tollway segments that have four lanes--two in each direction--90 percent of commercial travel is assigned to lane 1. For segments that have six lanes or more--three in each direction--98 percent of commercial travel is assigned to the driving lane. These values fall within the general limits suggested in the 1972 AASHTO interim guidelines for lane distribution on four- and six-lane highway facilities.

Axle Loads

At the AASHO Road Test, only loaded vehicles were used to impose a single value of axle load. Moreover, only the number of repetitions of loaded axles was counted. On the tollway, however, there exists a wide range of differing axle-load repetitions. It therefore becomes necessary to convert this distribution of axle load into a uniform axle-loading pattern. Since pavement distress increases exponentially with axle load, it should be noted that the equivalent axle load that represents the distribution will be different from and always greater than the mean axle load.

Individual axle loads for each vehicle type do not vary significantly with respect to time or geographic distribution. Therefore, the Illinois Department of Transportation axle-load information collected at loadometer stations throughout the state is used for summarizing 18-kip single-axle-load equivalencies per 1000 vehicles.

Statewide 18-kip equivalency factors for 1963, 1964, 1965, 1975, and 1977 for each vehicle class were used. A weighted mean equivalency factor was computed for each vehicle class by weighting the factor published for each year by the number of vehicles in that class per year. Loads imposed by class-1 vehicles are negligible compared with classes 2 through 6 and are omitted from the pavement axle-loading analysis.

For each vehicle class, the fraction of trucks was multiplied by the average 18-kip axle load in that class. These values were totaled for all classes to obtain an average 18-kip axle load per truck. This sum was then multiplied by the percentage of trucks in the driving lane to obtain the 18-kip axle loads in that lane per truck in the traffic stream. This result was the multiplied by the percentage of trucks in the traffic stream and by the directional traffic volume to obtain the number of 18-kip single-axle loads in the driving lane. The values obtained for all three periods were summed to obtain 18-kip axle loads for the entire study period.

Figure 6 illustrates an 18-kip single-axle-load history for the Tri-State Tollway between mileposts 0.0 and 6.0.

Future Traffic Loading

Cumulative 18-kip single-axle-load history curves (as illustrated in Figure 6) were used to estimate future axle loadings on each section of the tollway. These values were then used to establish serviceability histories to predict the total axle loadings for each pavement section to reach a TSI of 2.3, the value adopted by the tollway to schedule rehabilitation or overlays.

The more comprehensive the performance history, the more accurate the predicted performance of a pavement. In developing the service-life curves for the tollway system, the rating history consists of five serviceability points. The expected accuracy of projections based on this history will increase with time. Therefore, it is desirable that pavement performance be monitored by obtaining needed data to establish PSI values at regular intervals. This practice will improve the capacity to project future performance in addition to validating existing projections of service life. Further, major modifications in service-life curves may be required in the future to accommodate unanticipated changes in traffic patterns. This is more readily accomplished by using a complete up-to-date history of pavement performance.

For the tollway pavement sections, serviceability indices and traffic data were collected in 1967, 1969, 1971, 1975, and 1979 that provide five actual points on the service-life curve. Since more data that relate serviceability to cumulative 18-kip single-axle loads have become available, it is possible to construct service-life curves based on actual performance histories for each pavement section.

Figure 6. Cumulative axle-load history, Tri-State Tollway, mileposts 0.0-6.0.

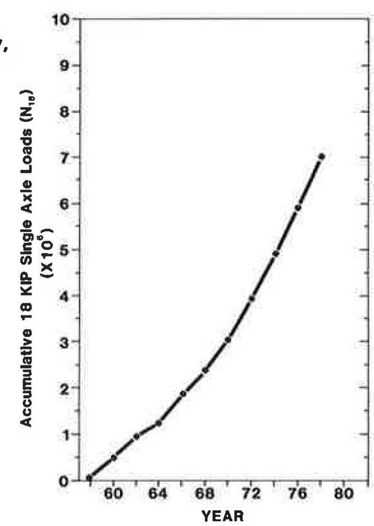
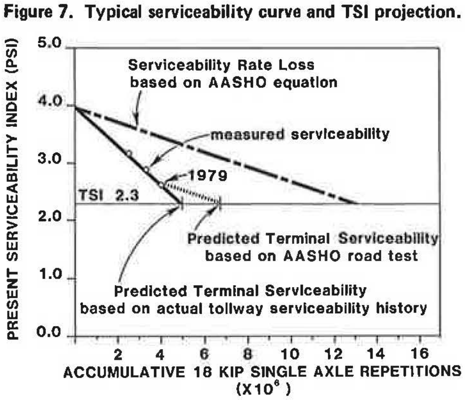


Figure 7. Typical serviceability curve and TSI projection.



SELECTION

The TSI is the point at which the pavement surface will not provide adequate service. The AASHO Road Test data revealed that when the serviceability index is reduced to 1.5, a pavement is completely unserviceable and would require reconstruction.

All previous tollway pavement evaluations have adopted 2.3 as the TSI to schedule rehabilitations or overlay measures. A TSI of 2.3 is retained for continuity of evaluation in this study.

SERVICE-LIFE CURVES

Figure 7 illustrates the service-life curve for a rigid pavement section on the Tri-State Tollway (mileposts 0.0-9.8). (Witczak has derived life curves for all pavement sections, and Figure 7 is typical.) By using both the AASHO Road Test method and actual performance history, times to TSI are determined. The AASHO method indicated that a serviceability value of 2.3 will require three times as many 18-kip single-axle loads as actual tollway serviceability trends indicate. By using the Tri-State Tollway performance data from milepost 0.0 to milepost 9.8, a TSI value of 2.3 will be obtained in two years. Actual serviceability loss on the Tri-State Tollway more closely fits the curve generated by the historical serviceability curves from the tollway. Therefore, the rate of serviceability loss as a function of cumulative 18-kip single-axle loads was derived from the serviceability trends established from 1958 to 1979.

REHABILITATION FORECASTS

The pavement service-life curves for each pavement section, for each direction of travel, and for all three tollways were completed. These curves are based on actual measurements through 1979. Projections from 1979 to a TSI of 2.3 were converted from equivalent 18-kip single-axle loads to time based on projected traffic volume and traffic mix. These data are summarized in Table 2 and provide the basis for the serviceability index.

Project limits are based on additional factors, which include location of toll plazas, pavement sections that have similar structural strengths, significant changes in traffic volume, significant changes in skid resistance, and similar typical pavement cross sections.

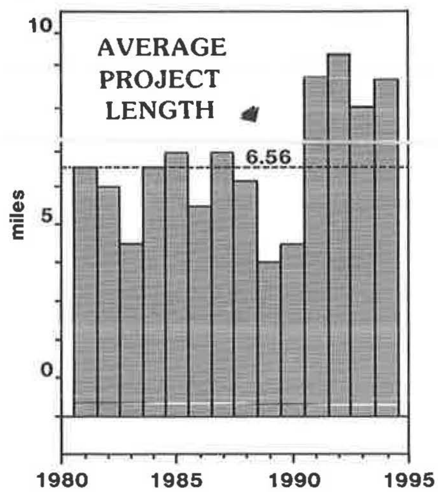
The resulting schedule is a 14-year cycle (Figure 8) in which major rehabilitation for all mileage will be performed. The average number of projects to be rehabilitated each year is between five and six; the average length is 6.56 miles. The cumulative percentage of system rehabilitation to be

Table 2. Proposed rehabilitation schedule.

Tollway	Milepost	Direction	PSI (1979)	Projected Year of Rehabilitation	Tollway	Milepost	Direction	PSI (1979)	Projected Year of Rehabilitation
Tri-State	0.00-9.18	NBL	2.64	1982		2.70-5.20		2.76	1983
	9.18-17.44		2.62	1982		5.20-10.60		2.75	1983
	17.44-23.30		2.76	1982		10.60-16.60		2.77	1983
	23.30-28.70		2.78	1982		16.60-23.26		2.86	1984
	28.70-31.50		2.57	1981		23.26-33.75		2.94	1987
	31.50-40.00		2.54	1981		33.75-39.21		2.86	1986
	40.00-46.00		2.69	1984		39.21-50.84		2.92	1987
	46.00-52.50		2.81	1985		50.84-54.06		2.96	1987
	52.50-55.60		2.98	1984		54.06-62.71		3.05	1988
	55.60-63.40		2.91	1984		62.71-67.49		2.95	1988
	63.40-70.00		2.90	1984		67.49-74.78		3.00	1989
	70.00-77.30		3.42	1991		74.78-76.31		3.00	1989
	0.00-9.18	SBL	2.81	1984	East-West	59.70-69.00	WBL	3.28	1992
	9.18-17.44		2.77	1983		69.00-82.00		3.43	1994
	17.44-23.30		2.76	1982		82.00-91.66		3.42	1994
	23.30-28.70		2.80	1982		91.66-94.00		3.45	1994
	28.70-31.50		2.68	1982		94.00-107.00		3.24	1991
	31.50-40.00		2.65	1981		107.00-117.00		3.25	1992
	40.00-46.00		2.67	1984		117.00-124.80		3.22	1991
	46.00-52.50		2.67	1984		124.80-128.90		3.32	1992
	52.50-55.60		2.74	1983		128.90-133.55		3.14	1990
	55.60-63.40		2.81	1983		133.55-138.15		2.17	
	63.40-70.00		2.76	1983		138.15-143.80		2.26	
	70.00-77.30		3.43	1991		143.80-149.70		2.99	1988
Edens Spur	48.20-53.30	WBL	3.20	1986		149.70-152.00		3.01	1989
	48.20-53.30	EBL	3.21	1985		152.00-156.00		2.94	1988
Northwest	0.00-2.70	WBL	2.72	1983		59.70-69.00	EBL	3.21	1991
	2.70-5.20		2.77	1983		69.00-82.00		3.45	1994
	5.20-10.60		2.63	1982		82.00-91.66		3.37	1993
	10.60-16.60		2.69	1982		91.66-94.00		3.47	1993
	16.60-23.26		2.97	1985		94.00-107.00		3.29	1992
	23.26-33.75		3.04	1988		107.00-117.00		3.38	1993
	33.75-39.21		2.95	1987		117.00-124.80		3.43	1994
	39.21-50.84		3.03	1988		124.80-128.90		3.44	1994
	50.84-54.06		3.05	1988		128.90-133.55		3.12	1990
	54.06-62.71		2.71	1984		133.55-138.15		2.15	
	62.71-67.49		2.95	1988		138.15-143.80		2.28	
	67.49-74.78		3.00	1989		143.80-149.70		2.93	1988
	74.78-76.31		3.00	1989		149.70-152.00		2.95	1988
	0.00-2.70	EBL	2.72	1983		152.00-156.00		2.90	1987

Figure 8. Tollway rehabilitation schedule.

Figure 9. Average project length for rehabilitation.



completed over the 14-year cycle and the average project length for each year are illustrated in Figure 9.

PROJECT DESIGN AND REHABILITATION ALTERNATIVES

Selection of rehabilitation type and corresponding project design to achieve the best value possible for funds expended and to provide smooth, safe, and economical pavements is done separately for each project. A study has been done by B. Ratteree of Crawford, Murphy, and Tilly, Inc., Springfield, Illinois, that provides an overview of those steps and additional data necessary for the project phase of pavement evaluation and management.

REFERENCE

1. AASHO Road Test: Report 5. HRB, Special Rept. 61E, 1962.

Publication of this paper sponsored by Committee on Pavement Rehabilitation Design.

Resurfacing of Plain Jointed-Concrete Pavements

HUGH L. TYNER, WOUTER GULDEN, AND DANNY BROWN

In 1975, the Georgia Department of Transportation placed a 1-mile concrete overlay test section on I-85 north of Atlanta, which has a high volume of truck traffic. The test area consists of 7.6-cm (3-in) continuously reinforced concrete (CRC), 11.4-cm (4.5-in) CRC, 15.2-cm (6-in) CRC, and a 15.2-cm (6-in) portland cement concrete (PCC) overlay. The primary objective was to determine the performance of various concrete overlay systems over a faulted jointed-concrete pavement. Some 16 asphaltic concrete overlay sections that had various thicknesses and treatments were placed adjacent to the PCC section in 1976. The performance obtained to date has indicated the importance of treatment of the existing pavement prior to placement of an overlay. Stabilization of moving slabs, replacement of fractured slabs, and patching and spall repair of the existing pavement are essential to the performance of the overlay. In addition, a level platform must be provided by grinding at the joints or by placement of a leveling course to prevent the overlay from being locked into the existing pavement by the faulted joints. Both 15.2-cm CRC and PCC sections, which have 4.6-m (15-ft) joint spacing, are performing well at this time. The 15.2-cm thickness of concrete overlay should be considered minimum for resurfacing over concrete when there is heavy truck traffic. The results from the asphaltic concrete test sections indicate that the use of a waterproofing membrane or fabric with a 10.2-cm (4-in) asphaltic concrete overlay will reduce the occurrence and the severity of reflection cracking from the underlying joints.

The Interstate system is nearing completion nationwide, and already many sections constructed 10 years ago or more are in need of major repairs or overlays. Many states are faced with the problem of how most effectively to upgrade existing plain jointed-concrete pavements that are suffering structural deterioration. The entrance of water through joints and cracks, the presence of erodible or compressible subgrade materials, and heavy load applications combine to cause nearly all distress in plain jointed-concrete pavements.

Yearly condition surveys made on Georgia's plain jointed-concrete pavements show that deterioration accelerates as the volume of truck traffic increases. Approximately 75 percent of the Interstate mileage in Georgia, or 1352 km (840 miles), is concrete pavement; 1078 km (670 miles) is plain jointed concrete.

Some of the older plain jointed-concrete pavements in Georgia are more than 15 years old, and many areas were in need of major repair or overlays. Many sections of Georgia's Interstate system have been rehabilitated or resurfaced or are currently being scheduled for upgrading. Since 1975, the Georgia Department of Transportation has initiated several research projects to find answers to the problems of rehabilitation techniques, water intrusion, overlay methods, and overlay thicknesses. The results of the concrete-overlay research project initiated in 1975 in Georgia are presented here and the asphaltic concrete-overlay test project will be discussed briefly for comparison purposes.

CONCRETE OVERLAYS IN GEORGIA

The first concrete-overlay project in Georgia was constructed in 1973. This project is a continuously reinforced concrete (CRC) overlay over an existing jointed-concrete pavement in the southbound lane of I-75 that extends from SR-42 near Forsyth to approximately milepost 175 near Macon for a total length of 21.9 km (13.6 miles). The original pavement consisted of portland cement concrete (PCC) 22.9 cm (9 in) thick that has expansion joints at 183-m (600-ft) intervals and contraction joints at 9.1-m (30-ft) intervals. This project is approximately 4.8 km (3 miles) long. The next 11.3 km (7 miles) of the original pavement is PCC 20.3 cm (8 in) thick that has 9.1-m joint spacing and expansion joints at 9.1-m intervals. The remainder of the original pavement section is a 25.4-cm (10-in) PCC pavement that has 9.1-m joint spacing over a 20.3-cm soil aggregate base in which the top 7.6 cm (3 in) is stabilized by using bituminous material. A CRC overlay 20.3 cm thick was placed from SR-42 to I-475 and had a steel content of 0.6 percent. From I-475 to the end of the overlay project, a distance of 4.5 km (2.8 miles), a CRC overlay 17.8 cm (7 in) thick was placed that had a 0.7 percent steel content.

No preparations were made to the original pavement and no attempts were made to bond the overlay to the original pavement or to provide for a positive bond breaker or stress-relief interlayer. Average daily traffic (ADT) levels on this section of I-75 currently are 30 000 ADT from SR-42 to I-475 and 13 500 ADT from I-475 to the end of the overlay.

The area that has the 17.8-cm overlay looks excellent; it has tight cracks and normal cracking patterns. The exception is near the bridge approaches, where the cracks appear to be somewhat wider and some closely spaced interconnecting cracking occurs. In these areas, the old concrete was removed prior to placing the CRC overlay to allow for a transition from the overlay to existing bridge decks.

The 20.3-cm CRC overlay in the area that has the higher traffic volume and that was placed over the 25.4-cm PCC pavement generally has normal cracking patterns that include fairly tight cracks. This section is generally in good condition; there are some Y-cracks and cluster cracking.

The cluster cracking is more pronounced and more extensive on the overlay section placed over the project that contained the 22.9-cm PCC that had the expansion joints. Some patching has been done in this area related to poor consolidation at construction joints. Wide transverse cracks are also present in this section and are thought to be related to the expansion joints, since the wide cracks are straight across the roadway and appear at regular intervals. The cluster cracking is probably occurring over the old joints in the original pavement. Overall, this project is in good condition. The overlay has recently been ground to restore the surface texture from SR-42 to I-475.

Research-Overlay Project

In 1975, an ad hoc committee that consisted of members from the American Concrete Paving Association, the Associated Reinforcing Bar Producers, the Portland Cement Association, and the Wire Reinforcement Institute published a report that described the results of a condition survey made on various CRC overlay projects nationwide. This survey showed that good results could be expected from CRC overlays that had a minimum thickness of at least 15.2 cm (6 in). Since no data were available on the performance of relatively thin CRC overlays, the Georgia Department of Transportation decided to place several concrete-overlay test sections that ranged in thickness from 7.6 cm to 15.2 cm.

A 1.6-km (1-mile) concrete-overlay test section was placed in November 1975 on I-85 in Gwinnett County 48.3 km (30 miles) north of Atlanta. This portion of I-85 was among the worst in Georgia in terms of faulting and broken slabs. The design characteristics and pavement condition of the existing pavement at the time of placement of the overlay are shown below:

Design Feature
- Pavement thickness: 22.9 cm
- Subbase: 20.3-cm soil aggregate; top 7.6 cm stabilized with cutback asphalt
- Joint spacing and design: 9.1-m undoweled
- Shoulder: cement-treated soil aggregate with asphaltic concrete shoulder

Performance
- Age: 15 years
- ADT
  - 1975: 17 200, 32 percent trucks
  - 1977: 20 000, 34 percent trucks
  - 1980: 21 500, 31 percent trucks
- Faulting: 2.5 mm or more, 83 percent; 5 mm or more, 29 percent
- Cracked slabs: 8 percent

This 1.6-km test section was divided into 400-m (0.25-mile) test areas and there was a short transition between each test area. The test sections selected were 7.6-cm CRC, 11.4-cm (4.5-in) CRC, 15.2-cm CRC, and 15.2-cm plain jointed-concrete pavement that used dowels and both 9.1-m and 4.6-m (15-ft) joint spacing.

Pavement preparation prior to overlay construction consisted of undersealing slabs, replacing broken slabs, and repairing and leveling the shoulder. The pavement was tested by using both static and dynamic loads and a Dynaflect, and slabs that experienced excessive movement were undersealed. Broken slabs were removed and replaced without the use of dowels. Curing compound was applied prior to paving in an attempt to break the bond between the old and the new concrete pavements.

Southbound traffic was diverted to the northbound lane through use of slip lanes across the median. Signing, delineation devices, and increased law-enforcement visibility were used; as a result, no accidents occurred during construction and traffic flowed satisfactorily.

During construction, measurements of concrete depth were taken frequently. In general, all aspects of construction of these test sections were closely monitored by research, laboratory, and construction personnel.

Construction of Test Section

The first test section placed was the 15.2-cm plain concrete pavement; dowels were placed over every old transverse joint. One-half of the section had joints sawed at 9.1-m intervals that matched the old joints, whereas the other half of the section had 4.6-m joint spacing. Transverse joints in this section that did not match an old joint were not doweled. The dowel bars were placed in baskets and were 2.9 cm (1.125 in) in diameter, 45.7 cm (18 in) long, and placed 38.1 cm (15 in) center to center. The dowels were coated with red lead paint for corrosion protection, and the dowel-bar assembly was anchored to the existing pavement. An open-cell neoprene joint sealant was used in the transverse joints. The longitudinal shoulders were sealed by using a hot-poured sealant.

The second test section placed was a 15.2-cm CRC pavement that had 34 no. 5 longitudinal bars, a total steel percentage of 0.6 percent. The steel was placed on chairs at the request of the contractors. The steel was lapped 50.8 cm (20 in) at the splices at a 30° angle. A 1.9-cm (0.75-in) transverse-doweled expansion joint was placed between the 15.2-cm plain concrete pavement section and the 15.2-cm CRC section.

The third test section placed consisted of 11.4-cm CRC pavement that had 40 no. 4 longitudinal bars, a total steel percentage of 0.6 percent. The bars were also placed on chairs in this section. A short transition section was placed between the 15.2-cm and the 11.4-cm CRC sections. The steel from both sections was carried through the transition.

The fourth and final section placed was a 7.6-cm CRC pavement that used 2.44x9.75-m (8x32-ft) woven wire-mesh mats for reinforcement. Three mats were placed over the two 3.66-m (12-ft) lanes. The welded wire-mesh openings were 10.2x30.5 cm (4x12 in) in D7.2 by D4 size steel. The welded wire fabric was lapped a minimum of 61 cm (24 in) over existing transverse joints. The wire mesh was supported on chairs spaced approximately 137 cm (4.5 ft) apart. A short transition section was placed between the 11.4-cm CRC section and the 7.6-cm CRC section. The reinforcing steel continued through the transitions to tie the CRC sections together.

The depth of the overlay for each section was controlled by a string line because of the settlement of many of the slabs. The actual overlay depth therefore is somewhat more than the design thickness in many areas.

Shoulder Construction

All sections contained concrete shoulders tied to the main-line pavement by tie bars spaced 76.3 cm (30 in) apart. No key was provided at the pavement edge. The width was 3.05 m (10 ft) for the outside shoulder and 1.22 m (4 ft) for the inside shoulder.

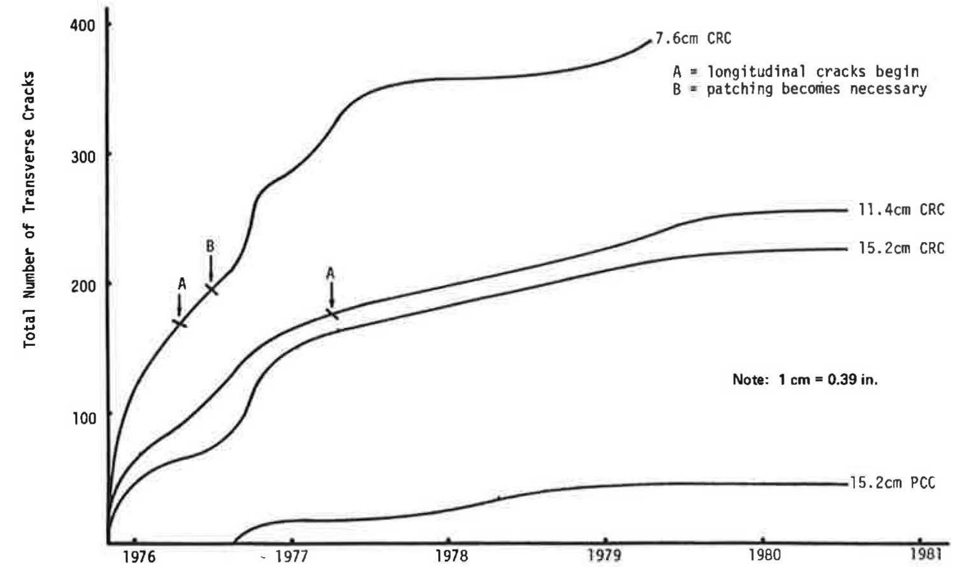
The 15.2-cm plain, 15.2-cm CRC, and 11.4-cm CRC sections had plain concrete shoulders and joints sawed at 9.1-m (30-ft) spacing that matched the location of the contraction joints in the old pavement. Rumble strips were provided in the shoulder to encourage the motorist to stay on the main-line pavement.

The shoulder in the 7.6-cm CRC test section was a welded wire-mesh reinforced shoulder tied into the main line by using tie bars. This shoulder also contained the rumble strips that were formed during the paving operation.

Construction Problems

Construction problems encountered on this project were confined to the 15.2-cm plain concrete overlay and the 7.6-cm CRC overlay. The dowels used in the plain concrete overlays were placed in baskets and attached to the existing surface by using nailed clips. Soon after construction had begun, it was evident that the basket assemblies were moving. At times a basket assembly would be displaced several centimeters and sometimes by as much as a meter. It was first thought that an insufficient number of clips was used to hold the basket assemblies down. Additional clips were added to the remaining basket assemblies. The additional clips seemed to stop most of the basket movement, but the outer two dowels on the baskets were still being moved forward. An inspection of the paving equipment revealed that the minimum opening of the paver was 716.3 cm (23.5 ft) and that the basket assemblies were 716.3 cm wide. The basket assemblies had been contacting the paver, which pulled them forward. Adjustments were made, and no further problems with dowel-basket movement occurred. However, this adjustment was not made until the 15.2-cm plain concrete test section had been nearly completed.

Figure 1. Crack progression in concrete-overlay test project, I-85, Gwinnett County.



Some problems were encountered with the placement of the 7.6-cm CRC test section. These problems were minor and caused no structural damage to the overlay. The combination of thin overlay depth and placement of both reinforcing mesh and shoulder tie bars at middepth caused these problems. The reinforcing mesh and shoulder tie bars had to be carefully arranged to maintain adequate concrete cover.

Performance of Concrete Overlay

The performance evaluation of the test section consists of visual observations of the condition, mapping of all cracks, deflection measurements, and movement measurements across the joints and cracks in the overlay sections at regular intervals. The appearance of the cracks and the cracking patterns have given the best information with respect to performance to date. Deflection measurements are highly dependent on temperature; deflections obtained early in the morning are generally much higher than those obtained later in the day, due to curling of the underlying pavement.

Hairline cracking appeared in the CRC overlay sections several days after placement of each section. These hairline cracks were always located directly over the old contraction joint and occurred over each construction joint with no exceptions. The progression of additional cracking always occurred near the joints within 30-60 cm (12-24 in) of the original crack. A crack survey was conducted two months after completion of the project; it showed that in the 7.6-cm CRC section, multiple cracking of two cracks or more was present over 65 percent of the old construction joints compared with 50 percent for the 11.4-cm section and 34 percent for the 15.2-cm section. The occurrence of cluster cracking as well as the number of cracks in the cluster decreased with the thickness. This trend was reinforced during subsequent performance evaluations. The progression of the total number of cracks in the CRC test section can be seen in Figure 1.

As the CRC cracking progressed, the next cracks tended to occur approximately midway between the underlying joints (midslab). These cracks emanate from the sawed centerline joint and progress to the shoulder.

Further crack progression results in multiple transverse cracks near the underlying joints, additional midslab cracks, and the development of longitudinal cracks. Short longitudinal cracks develop that interconnect the multiple transverse cracks over the underlying joints and generate punchouts in the 7.6-cm CRC test section.

The 7.6-cm CRC section has progressed through all the steps mentioned above. Extensive cracking, punchouts, and patches exist in the 7.6-cm CRC section. Approximately 20 percent of the areas over the underlying joints in the 7.6-cm CRC section have required patching.

It was evident during the patching operations that all the transverse and longitudinal cracking occurring in this section was located over the steel in the reinforcing mats. This fact indicates excessive stress from the concrete, probably caused by deflection at the joints of the underlying PCC pavement.

The 11.4-cm CRC section has multiple cracks over the underlying joints, midslab cracks have progressed, and longitudinal cracks have appeared and are progressing. No major patching has been necessary, but some edge punchouts are present in this section, which indicates initial structural failures.

The 15.2-cm CRC overlay has shown very good performance to date. One or two cracks occur soon after construction and are associated with the underlying joint. Midslab cracks occur, but multiple cracking near the underlying joint and longitudinal cracks are minimal.

The 15.2-cm PCC overlay that has 9.1-m joint spacing has had cracking in approximately 65 percent of the overlay slabs. This high percentage of slab cracking could be attributed to inadequate bond breaking between the overlay and the existing pavement. Differential slab curling between existing and overlay pavement could cause the cracking.

The 15.2-cm PCC overlay that has 4.6-m joint spacing has had 30 percent of the overlay slabs cracked or broken. The majority of the cracked overlay slabs that have 4.6-m joint spacing occur at the beginning of the project, where problems occurred with movement of the dowel-basket assembly.

ASPHALTIC CONCRETE OVERLAYS

We are mainly concerned here with the performance of unbonded concrete overlays for jointed-concrete pavements. For comparison purposes, a description of the design and performance of the asphaltic

Table 1. Cracking in overlay, southbound lane, I-85, Gwinnett County, June 1980.

Treatment Before Overlay[a]	Asphalt Concrete Overlay: 5.1 cm, Reflection Cracking (%)	5.1 cm, Severity of Cracking[b] (%)	10.2 cm, Reflection Cracking (%)	10.2 cm, Severity of Cracking (%)	15.2 cm, Reflection Cracking (%)	15.2 cm, Severity of Cracking (%)
Bituthene	87	44	57	17	0	0
Mirafi	100	80	60	24	4	1
Petromat	98	71	57	22	18	4
Edge drain	100	93	98	78	66	29
Control	100	98	95	62	24	7

Note: 1 cm = 0.39 in.
[a]Arkansas base (16 percent and 6 percent) was also used.
[b]Length of transverse cracking reflected through overlay as percentage of total length of transverse joints.

concrete test sections located adjacent to the PCC sections is now included.

Sixteen asphaltic concrete overlay test sections were placed in 1976 to compare the performance of this type of overlay with the performance of the CRC and PCC test sections. The major variables in the asphalt test sections were three overlay thicknesses--5.1 cm (2 in), 10.2 cm, and 15.2 cm--and various treatments prior to placement of the overlay. These treatments consisted of the placement of two different engineering fabrics (Mirafi and Petromat), the addition of edge drains, the placement of a stress-relieving interlayer referred to as "Arkansas Base" that consists of large, one-size stone held together by using bituminous liquid, and the placement of strips of a heavy-duty waterproofing membrane (Bithuthene) over all existing joints and cracks. Control sections in which none of the above treatments was used were also placed with each overlay thickness. All treatments were repeated with each of the three overlay thicknesses. A 372-$N \cdot m^2$ (70-$lbf \cdot yd^2$) leveling course was placed in addition to the overlay.

The performance is evaluated mainly in terms of the number and severity of reflection cracks from the existing joints into the overlay. These data are shown for June 1980 in Table 1, which indicates that the various treatments have had a significant effect in reducing the rate of occurrence of reflection cracking and in reducing the severity of the cracking, especially with the 10.2-cm and 15.2-cm thicknesses. Based on the early results from these test sections, the Georgia Department of Transportation has for the past three years been including the heavy-duty waterproofing strips in all projects on the Interstate system where asphaltic concrete overlays (normally 10.2 cm) were placed over existing jointed PCC pavement.

TREATMENT OF EXISTING PAVEMENT

The results to date of the test sections point out that the preparation of the existing pavement prior to placement of the overlay is of utmost importance if excellent performance of the overlay, whether it is asphalt or concrete, is to be obtained. The various treatments that can be used to prepare the existing pavement are (a) stabilizing moving slabs by means of undersealing, (b) addition of edge drains, (c) replacement of fractured slabs, (d) patching of spalls, and (e) resealing of open or wide joints and placement of a waterproofing membrane if an asphalt overlay is to be used.

One of the most important factors in the success of an overlay is to provide a stable platform on which to place the overlay. If this is not done, distress in the form of excessive cracking and eventual punchouts will occur in a short time in the overlay. When a slab is stabilized, it must be recognized that lifting must be avoided in order not to create voids in another area of the pavement. All fractured slabs should be replaced, since they represent a structural failure in the pavement and will likely create problems in the overlay in the future. In the same manner, spalls and small failures should be repaired if the distress is severe. Under no circumstances should it be relied on for the overlay to bridge over problem areas in the existing pavement if an economical overlay thickness is to be used. Edge drains are frequently added to a pavement as a rehabilitation measure to remove infiltrated surface water. The experience in Georgia to date on rehabilitation projects has showen that edge drains are not effective on a long-term basis for the prevention of pumping and faulting. The test sections on the I-85 overlay project that used edge drains have a significantly higher incidence of joints that show reflection cracking than do the other sections for the 15.2-cm asphalt concrete overlay area (Table 1). This difference is not as apparent in the sections that use the other overlay thicknesses because of the high percentage of reflection cracking that is present in all these test areas.

CONCLUSIONS

1. The performance of the overlay on I-75 shows that a thick, partially bonded CRC overlay can be placed over a PCC pavement with excellent results. The thickness of the overlay, however, may not be economically feasible for most projects.

2. The results to date of the test project on I-85 indicate that in order to obtain good performance of a relatively thin unbonded concrete overlay, the existing pavement must be properly prepared (undersealed, broken slabs replaced, patched, etc.). Furthermore, placing curing compound as a bond breaker when faulted joints are present is not sufficient since the overlay is locked into the expansion and contraction movements of the underlying joints. The overlay should be placed on a flat horizontal plane, which can be established by grinding the joints flush or by placing an asphaltic concrete leveling course. A bond breaker should then be placed prior to placement of the overlay.

3. Multiple cracking in the CRC overlay over the existing joints will occur if moving slabs are not stabilized. All distress found on the 7.6-cm CRC overlay section is occurring over old joints. This type of distress is also likely to occur in time with thicker CRC overlays.

4. The performance of the concrete shoulders on the overlay project has been excellent. Concrete shoulders should be used when a concrete overlay is placed.

5. The 7.6-cm CRC section has not provided acceptable performance and is not considered a

suitable rehabilitation measure for plain jointed PCC pavements.

6. From the current condition of the test section, the 11.4-cm CRC section could perform acceptably for up to 10 years with some maintenance. This overlay design could be used successfully on sections that have moderate traffic levels if the existing pavement was properly prepared.

7. The minimum thickness for a concrete overlay on a road that has a large volume of heavy trucks should be 15.2 cm. Both the CRC and jointed PCC sections that used the 15.2-cm thickness are doing well after five years of heavy truck traffic. From the performance of the two PCC overlay sections on I-85, the joint spacing in a 15.2-cm jointed PCC overlay should be 4.6 m. All the joints in the original pavement should be matched in the overlay and intermediate joints added to obtain the desired joint spacing. The transverse cracking that occurs on the test sections is attributed to the movement of the dowel assemblies and other start-up problems at the time of construction of these short test sections. These problems are not expected to occur on a regular construction project.

8. The major problem with using concrete overlays is the necessity of closing the roadway and diverting the traffic for an extended period of time. Generally the traffic will have to be directed onto the adjacent travel direction, which requires temporary concrete median barriers and upgrading and widening of the shoulders to maintain two lanes of traffic in each direction on an undivided highway or construction of slip ramp on a divided highway. This additional expense must be considered in determining the cost of a concrete overlay. From the standpoint of accidents and traffic flow, the slip ramps used on the research-overlay project performed very satisfactorily. This performance was felt to be due to proper design of the slip ramp to minimize loss of speed by the driving public, to signing, and to delineation between opposing lanes of traffic.

9. The experience in Georgia to date on the overlay sections reported here and on other rehabilitation projects has shown that edge drains are not effective on a long-term basis for the prevention of pumping and faulting.

10. In the asphaltic concrete overlay sections, the placement of strips of heavy-duty waterproofing membrane (Bithuthene) over all joints and cracks in the existing PCC pavement has proved to date to be the most effective method of reducing the number and severity of reflection cracks from the existing joints into the overlay. Engineering fabrics (Mirafi and Petromat) and the stress-relieving interlayer (Arkansas base) were also effective. The edge-drain treatment was not effective; in fact, it was worse than no treatment (control sections).

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Design Procedure for Premium Composite Pavement

W. RONALD HUDSON AND FREDDY L. ROBERTS

A brief description of a method for designing premium composite pavements is given. A premium pavement is defined as a pavement structure that will perform free from structural maintenance for 20 years and will require only minimum maintenance for an additional 10-20 years. To satisfy this requirement, the procedure incorporates the best current design and construction practices for composite pavement. The method described provides highway engineers with a systematic technique for selecting a premium pavement design. The procedure is more complex than some empirically based systems but is relatively easy to use. The complete manual includes a number of charts, figures, and worksheets and a procedure for their use. On the basis of design data, a user can select a precalculated pavement cross section from a catalog of designs. An overview of the design system and a brief explanation of the procedure and of the factors considered in the development of the procedure are given. An explanation of the design inputs required for use of the procedure is also included. A discussion of the limiting criteria used in development of the design procedure for premium pavement is followed by special considerations required for the design of reinforcement in the concrete layers of composite pavements and discussion of the design methodology.

For several years, the Federal Highway Administration (FHWA) has pursued multiple research studies aimed at producing premium pavement structures for heavily traveled highways. The intention of these efforts has been to develop pavements and minimize maintenance, which disrupts traffic flow and creates hazards and high user costs. This research is aimed at the development of pavement structures that will be maintenance-free for a minimum of 20 years and will require only routine maintenance for 10-20 years thereafter. A composite pavement has, in this case, been defined by FHWA as a pavement made up of both rigid and flexible layers and that has an asphaltic surface layer. The rigid layer(s) may be portland cement concrete (PCC) or cement-treated soil or base.

The research reported here is drawn from a portion of an FHWA-sponsored research project on flexible and composite structures for zero maintenance. The overall goal of that project is to develop pavement design procedures that can be used to design the thickness, specify the materials, and specify the unique construction procedures required for premium flexible and composite zero-maintenance pavements.

In using the design method for composite pavements discussed here, we emphasize that sound engineering must also be used in selecting any pavement design strategy. The designer must recognize that it is difficult in a general design procedure to successfully couple the knowledge of performance of local materials and the service requirements for

Curling Stress

One item that should be considered in the performance of composite pavements is the thickness of asphalt concrete required to significantly reduce the curling stress in the concrete slab. Basically, the temperature differential of the concrete slab is computed as a function of asphalt concrete thickness. For the majority of the conditions investigated, 3 in of asphalt concrete was found to significantly reduce the temperature differential of the concrete slab and the resulting curling stress and is included as a minimum thickness in this procedure.

Low-Temperature Cracking

The low-temperature cracking criterion is initially used to define the material properties for the asphalt concrete or continuously reinforced concrete (CRC). Both programs TC-1 (6) and CRCP-2 (7) were used to define the materials necessary to achieve proper performance. Regression equations were developed for use in the design of the required percentage of steel for the CRC rigid base layers instead of the CRCP-2 program. Once the material properties have been defined, the thicknesses to resist fatigue cracking and rutting can be determined.

Fatigue Cracking

This step provides for the determination of the thickness of the rigid layers required to limit the amount of fatigue cracking. A series of plots was developed to allow for a graphical determination of the required low-modulus concrete thickness for composite pavements by using results from VESYS III (8) and SLAB49 (9).

Reflection Cracking

The computer program RFLCR-1 (10) was used to determine whether reflection cracking was predicted to occur in composite pavements. Based on numerous studies, it was concluded that an asphalt crack relief layer provided the best potential for minimizing the distress manifestation of reflection cracking.

Deflection and Subgrade Compressive Strain

By using the FHWA criteria (1) and the computer program VESYS III, surface deflection and subgrade compressive strain were computed for pavement cross sections that satisfy the previous criteria. If the pavement structure does not meet the criteria established, then stabilized materials are used to reduce these two pavement-response variables.

Frost Penetration

Once the pavement structural thicknesses have been determined, frost action must be considered. By using the procedure described by Barber (11), the frost penetration is predicted, if a frost-susceptible material exists. If the frost penetrates a frost-suspectible material, then additional thicknesses or an insulating material must be used to prevent frost heave or spring-thaw strength reductions. Detailed guidelines for determining the treatment method are supplied in the design manual (2), which includes a series of figures prepared to simplify this step of the procedure.

Summary

The procedure is an iterative one in which there is much interaction between user and model. For the procedure to design premium pavements accurately, the user must have adequate information concerning the materials in a specific environment. Without this information, no model or procedure can be expected to design for proper performance reliably. It is also noted that if the user elects to use the graphs presented later in this paper instead of each computer program, it must be ensured that the simplifying assumptions have been met.

DEVELOPMENT OF LIMITING CRITERIA

We next describe the selection of the limiting criteria for each of the significant distress checks made as a part of this design procedure.

Fatigue of PCC

One of the major distress mechanisms associated with rigid pavements is fatigue cracking, defined by Mills and Dawson as "the process of progressive localized permanent structural change occurring in a material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in cracks or complete fracture after sufficient number of fluctuations" (12). The use of cracking alone as a design criterion is inadequate, since the formation of a crack does not necessarily imply a functional failure, as has been reported by several investigators (10,13-16). These fatigue relationships were determined from data collected on reinforced and nonreinforced jointed concrete pavements for different levels of service [class 3 and 4 cracking and a present serviceability index (PSI) of 2.5]. In order to develop a fatigue failure criterion for zero-maintenance pavements, the specific effects of four factors on fatigue cracking of concrete layers were investigated. These factors were failure criterion, pavement type, concrete type, and laboratory data.

Failure Criterion

In establishing failure criteria, it becomes questionable whether fatigue relationships based on concrete surface layers are valid for composite pavements, since the concrete layer is no longer the riding surface. Hence fatigue criteria may be different if based on PSI values for composite pavements. Failure criteria based on cracking also may be different for composite pavements, but the difference should be much smaller, since cracking is primarily related to concrete stress and strength magnitudes, whereas PSI is related primarily to roughness. Since performance of a composite pavement is directly related to the cracking of the rigid base layer, cracking was selected as the failure criterion for the concrete layer.

Pavement Type

Several fatigue relationships based on the performance of reinforced and nonreinforced jointed concrete pavements were studied. In studies of the AASHO Road Test completed by Treybig and others (10) and by Yimprasert and McCullough (17), it was observed that essentially no difference existed in the sections that failed, although the nonreinforced sections had a greater probability of survival and shorter time span between the different levels of cracking. Therefore, from these analyses and since no other studies reported comparisons of fatigue cracking in reinforced and nonreinforced pavements, the same fatigue-cracking criteria were applied to all types of concrete pavement. Only a difference in loading conditions (edge, corner, or interior) based on pavement type was used to predict the applied flexural stress in the concrete layer.

Concrete Type

The fatigue relationships are generally based on the road-test concrete pavements but are based on different failure definitions, except for the curve developed by Treybig, McCullough, and Hudson (13), which is based on failures of a jointed concrete pavement at airfields. Therefore, it is unclear whether these same fatigue relationships can be applied to other concrete mixtures such as lean concrete or econocrete. Laboratory fatigue results by Raithby and Galloway (18) that illustrate the effect of curing time and mixture properties on the fatigue relationships show that all mixtures exhibit approximately the same relationship between the stress/strength ratio and cycles to failure. Laboratory fatigue results for specimens that range from PCC to lean concrete show that there is no distinct relationship for each type of concrete; i.e., the relationship for lean concrete falls between other relationships determined for PCC (18-21; unpublished report by C.E. Kesler on fatigue and fracture of concrete, University of Maryland). The loading frequency of concrete beams does not have a great effect on the fatigue life of concrete (18). Therefore, it has been assumed that one fatigue curve can be used to describe the cracking failure criteria for all types of concrete mixtures. However, this might not be true if the concrete were the surface layer and failure were based on PSI.

Laboratory Data

Results from laboratory studies compared with the fatigue characteristics determined from field pavement performance show a difference. For large stress/strength ratios, a larger number of load applications are expected before terminal serviceability is reached in the field than is observed in the laboratory. This is due to the interval between initial cracking and the severity level of cracking defined as pavement failure. For small stress/strength ratios, a smaller number of load applications are expected before terminal serviceability is reached in the field than is observed from laboratory results. This is probably the result of environmental factors, the dynamic effect of loads, and other damage factors that may have been underestimated in developing the field fatigue curves. Therefore, until more-reliable fatigue information can be obtained, the fatigue curves that describe field performance will be used to determine fatigue failure for low-modulus concrete.

To summarize, in this method the failure of the concrete layer for composite pavements was based on cracking severity determined from pavement performance data. By using this failure criterion, zero maintenance should not be required for the traffic specified in a 20-year period, during which time cracking will cause a pavement condition in which routine maintenance will be required to provide additional years of service.

Transverse Cracking

There are almost no data available to establish a failure criterion for transverse cracking (including low-temperature and reflection cracking). Based on observations of in-service pavements, Darter and Barenberg established a 10- to 30-ft crack spacing as a limiting value (22). Therefore, based on this information and the project staff's experience, an average crack spacing of 30 ft was selected as the failure criterion for transverse cracking in the surface layer.

For the concrete slab in composite pavements, the CRC layer's performance has also been observed to be related to crack spacing. Based on previous observations and performance studies, a crack spacing greater than 4 ft but less than 8 ft has been observed to provide adequate serviceability.

Deflection

Surface deflection has also been related to field performance by a large number of researchers. In fact, a large number of design and evaluation procedures are based on surface deflection. Therefore, surface deflection was selected as an additional limiting design criterion for zero-maintenance pavements. This allows the use or consideration of data from a large number of studies and observations of past pavement performance. Therefore, based on a review of existing data, Figure 3 (23) was selected to develop the pavement structural cross section to limit surface deflections.

Roughness

The other important performance parameter used in establishing limiting criteria for zero-maintenance pavements is roughness or PSI. Based on previous studies conducted on flexible and composite pavements, a PSI lower than 3.0 will require maintenance. Hence, a limiting PSI value of 3.0 was selected in establishing the pavement cross section to meet the zero-maintenance criteria.

SPECIAL CONSIDERATION OF REINFORCEMENT

Some reinforcement will be required for concrete layers used in construction of composite pavement. For jointed concrete subsurface layers, load-transfer devices are suggested for most situations to prevent potential faulting and subsequent reflection of cracks to the surface. In CRC subsurface layers, longitudinal steel runs continuously throughout the length of the pavement. In CRC subsurface layers, transverse reinforcement is provided for most pavements. Other uses of reinforcement occur at terminal anchorages, construction joints, and edges of pavements that have tied shoulders, as previously discussed. Space does not permit the treatment of reinforcement here (1,2).

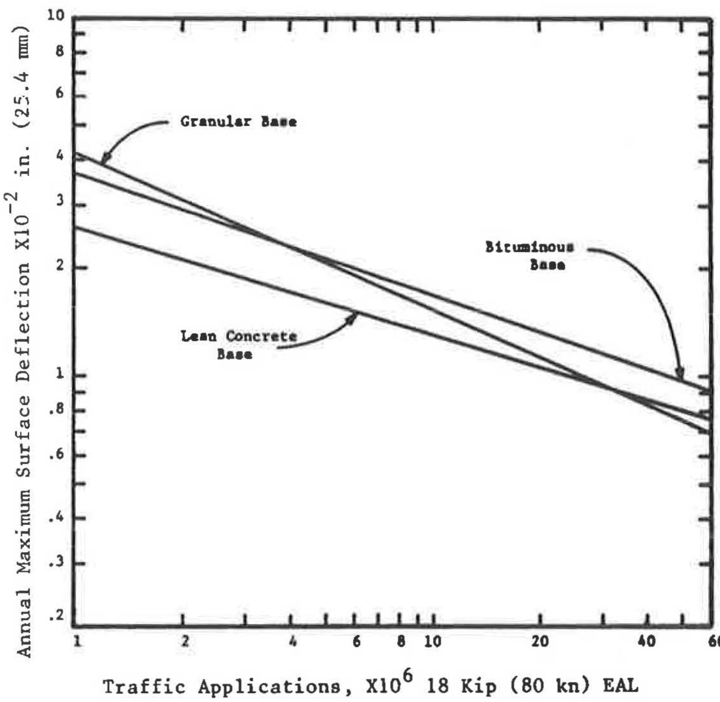
THICKNESS DESIGN PROCEDURE

The structural design charts have been prepared in a form in which combined traffic applications and PCC flexural strengths are used to determine layer thicknesses. A set of charts has been prepared that are used to determine the principal thicknesses for PCC layers. The charts determine the structural thickness required to satisfy the design criteria of fatigue cracking, rutting, and roughness. A second set of charts has been developed to determine the requirements for stabilized base thickness as a function of traffic and subgrade modulus and to indicate the additional thickness required to satisfy the deflection and subgrade vertical compressive strain criteria. A complete treatment of the procedure is contained in the FHWA report (1). Only certain sample charts are included in this paper.

Design Inputs

The designer must collect the basic information described in the following paragraphs.

Figure 3. Allowable maximum surface deflection used to develop structural cross section for each pavement type.



Environmental Factors

Temperatures are used to determine depth of frost penetration. The temperature values required are (a) mean annual air temperature, (b) warmest mean monthly air temperature, and (c) the coldest mean monthly air temperature. These air temperatures may be obtained from climatological records (24). They are used to reflect the effects of other environmental factors, such as solar radiation, daily temperature variations, and minimum design temperatures for specific areas in the United States. We have developed charts for obtaining the number of the environmental region as well as for predicting the annual minimum pavement temperature for each region (1,2).

Natural Soil Properties

The subgrade modulus is used as one of the principal design parameters. Since the modulus of most subgrade soils is dependent on the state of stress, the subgrade modulus should be based on in situ or expected field conditions, e.g., a moisture content and density that the soil is likely to reach under the pavement structure. This equilibrium moisture content is usually similar to that found at a depth of about 3 ft in the natural soil.

By using information on soil properties from classification tests, the engineer must determine whether stabilization of the existing subgrade is required. The type and amount of stabilizer should be substantiated by approved testing methods.

Traffic

Traffic is usually expressed in 18-kip equivalent single-axle loads as presented in the American Association of State Highway and Transportation Officials (AASHTO) Interim Guide (25) and as developed at the AASHO Road Test (26). In this design manual, total traffic is also expressed as the number of 18-kip equivalent axle loads that occur in the design lane during the minimum 20-year maintenance-free design life.

Thickness Determination

By using the total projected traffic, low-modulus PCC flexural strength (28 days), and the subgrade modulus, the thickness can be determined for each pavement type. For all pavement cross sections that require a stabilized base layer, an improved subgrade-layer thickness of 24 in is required. The minimum thickness recommended for the stabilized base layer is 6 in.

The minimum asphalt concrete thickness required over a CRC rigid base is 3 in, to minimize curling stress. The asphalt concrete material placed over a jointed concrete rigid base consists of 3 in of an asphalt crack relief layer and 2 in of dense graded asphalt concrete. With this surface thickness, the relationships shown in Figures 4 and 5 (1) are used to determine the thickness of low-modulus concrete required to resist fatigue cracking and retain an acceptable PSI level. The selection of a particular design chart is based on the 28-day concrete flexural strength determined by ASTM C-78 or AASHTO T-97.

Figures 6 and 7 (1) can be used to determine the thickness requirements for stabilized base layers of asphalt or cement in combination with the low-modulus concrete base for composite pavements.

Adjustment for Frost Susceptibility

The frost penetration below the pavement surface can be estimated for the pavement cross section by using the natural soil type and the environmental region in a relationship shown in Figure 8 (1) for region 2. If the subgrade soil is frost-susceptible, two design alternatives are possible. The engineer may elect either to totally protect the subgrade from frost penetration or to increase the structural thickness to account for reduced strength values that occur during spring thaw. Suggestions are provided in the manual (1,2) to assist the engineer in choosing between full and partial protection from frost penetration. Among the factors that affect this decision are soil classification, strength dur-

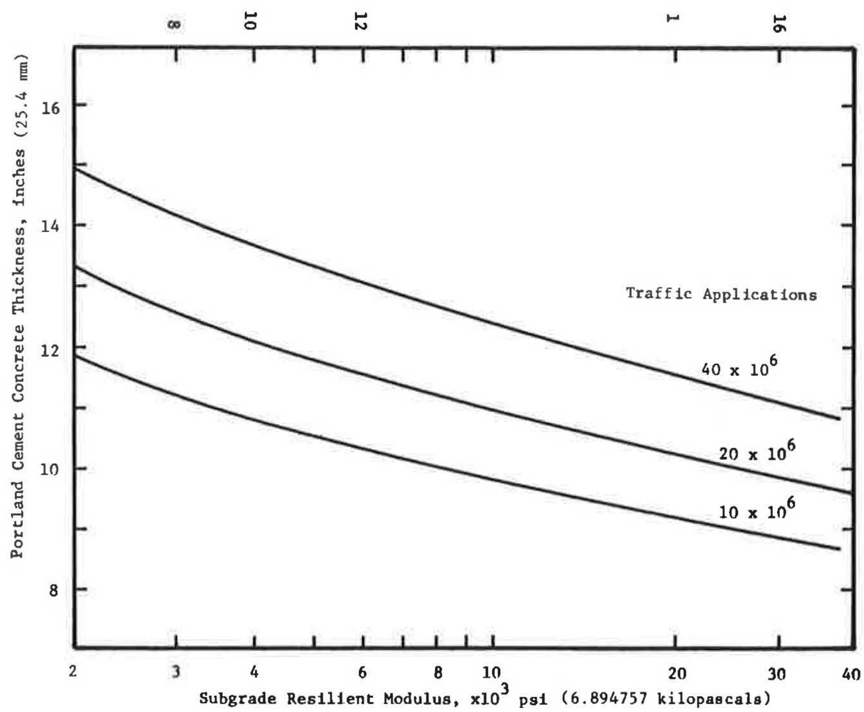


Figure 4. Low-modulus CRC thickness required by fatigue and roughness criteria for composite pavements with flexural strength of 500 lbf/in^2.

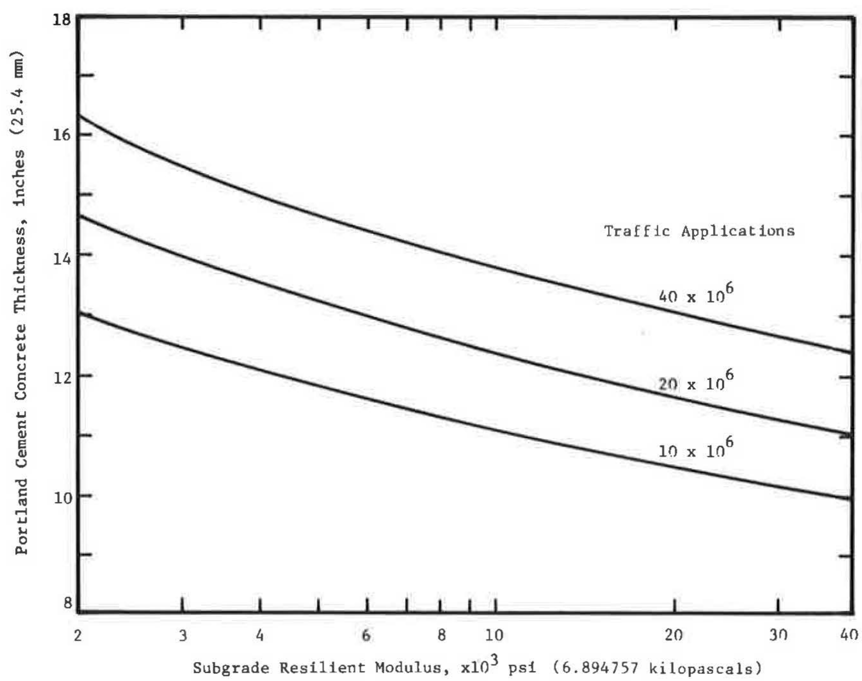


Figure 5. Low-modulus jointed concrete pavement thickness required by fatigue and roughness criteria for composite pavement with flexural strength of 500 lbf/in^2.

ing the thaw period, water-table location, pavement type, and soil modulus.

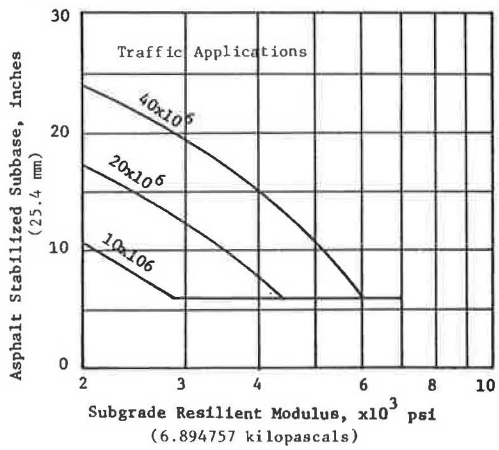
If the engineer elects to protect the subgrade fully from frost penetration, relationships similar to that shown in Figure 8 should be used to estimate the additional thickness of non-frost-susceptible materials required as a part of the pavement structure. A decrease in the stiffness of subgrade soils due to spring thaw will result in a reduction of the stiffness of the granular materials, which may not be recovered with time.

If the engineer desires to analyze the partial-protection alternative, relationships similar to the ones shown in Figure 9 (1) should be used to estimate the increased damage that occurs as a result of reduced strength during the spring thaw. After evaluation of both the partial-protection and full-protection alternatives, the engineer can select the most cost-effective design.

SUMMARY AND CONCLUSIONS

The materials included in this paper identify and examine the factors and requirements for establishing zero-maintenance pavements. We have examined the primary distress manifestations and correspond-

Figure 6. Stabilized subbase thickness of asphalt required to meet surface deflection and subgrade compressive strain criteria.



ing distress mechanisms that govern the behavior of composite pavements. Materials and material properties were reviewed in relation to providing the maximum performance required by the definition of premium pavement.

From this information and experience, candidate premium pavement cross sections were established that have strong potential to function for 20 years or more with minimum maintenance. A unique catalog of designs was chosen to present the resulting cross sections. Examples of these results are shown in Figures 10 and 11. Specific models were selected that, in the opinion of the research team, have the highest potential reliability for predicting actual performance. By using these models and past performance information, design criteria for each distress manifestation were established for design of composite structures. Subsequently, a design procedure was developed and organized so that both environmental and traffic-induced damage would be considered.

Figure 7. Stabilized subbase thickness of cement required to meet surface deflection and subgrade compressive strain criteria.

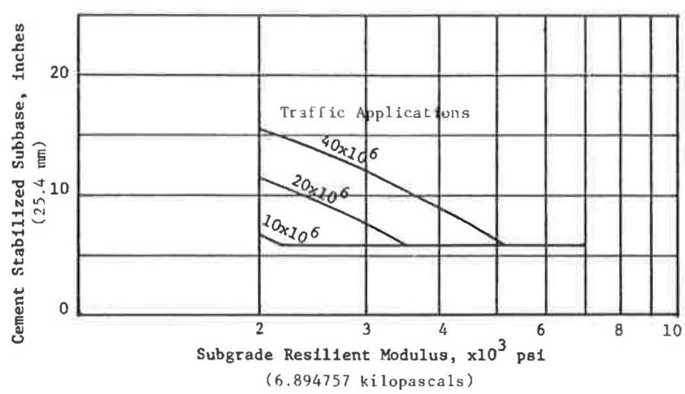


Figure 8. Determination of frost penetration into different materials below pavement surface for environmental region 2.

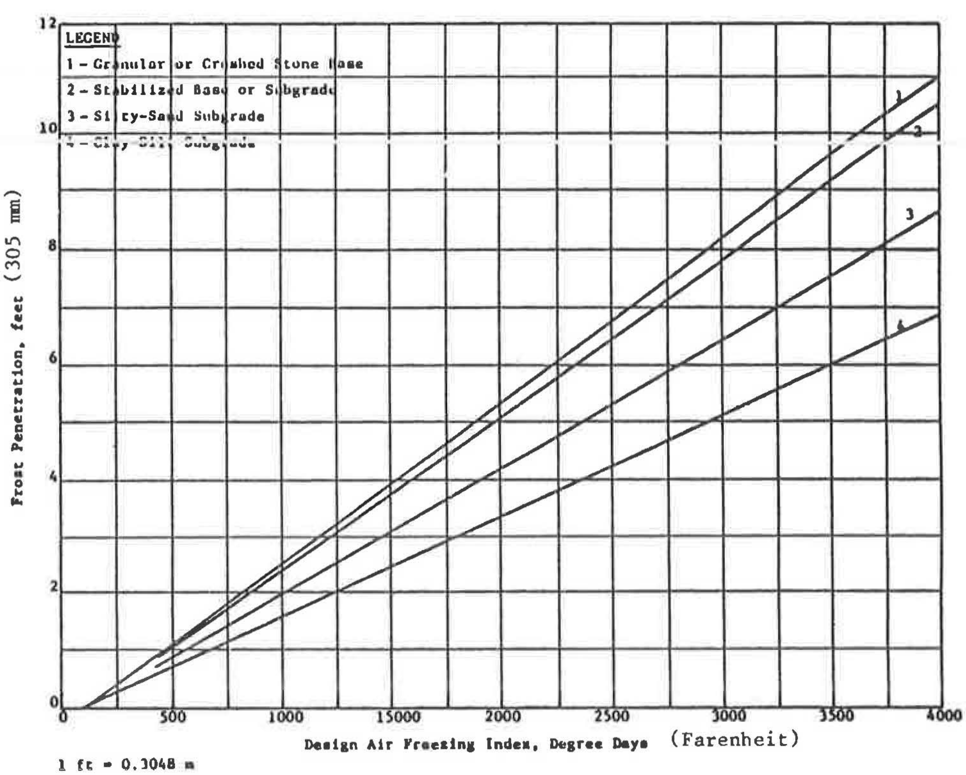


Figure 9. Increase in structural thickness of composite pavements for different levels of frost damage.

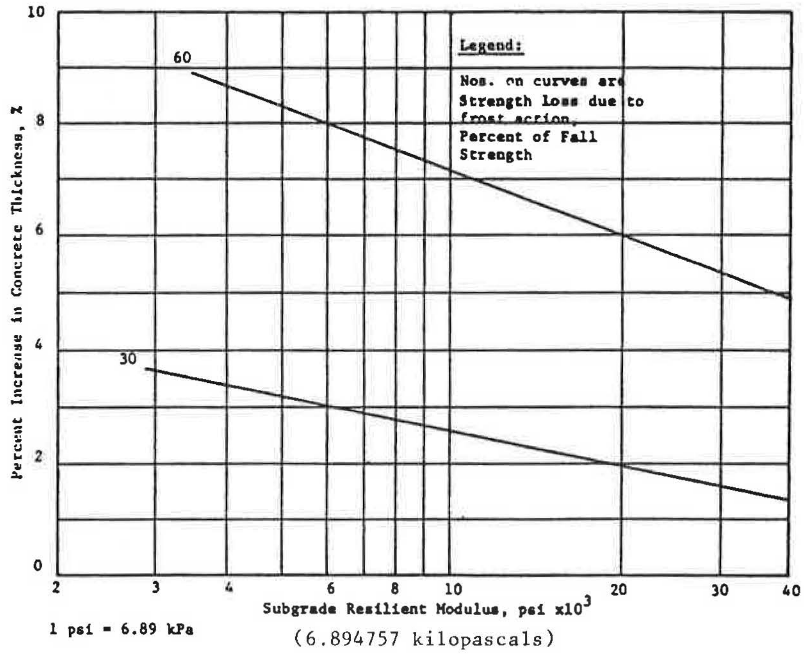
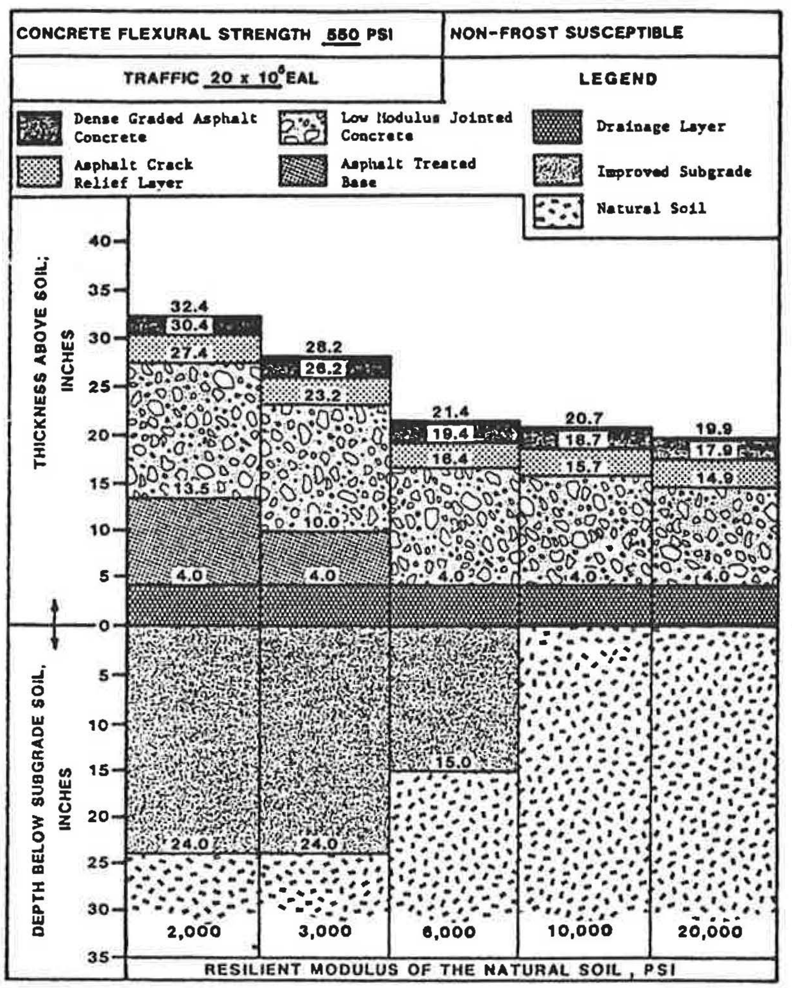
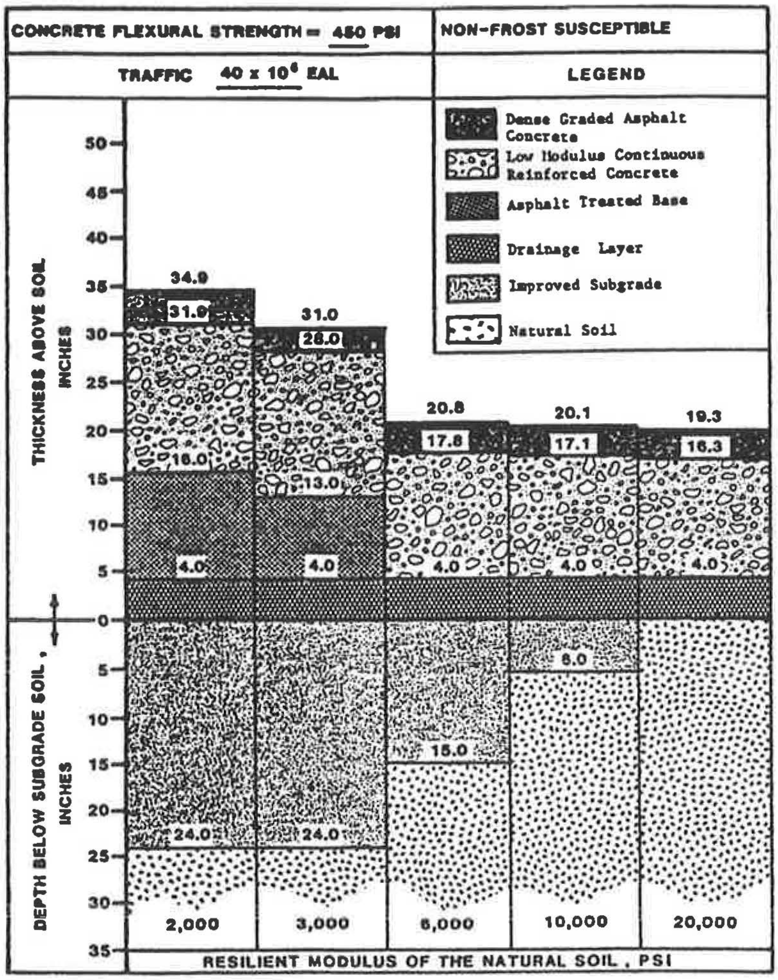


Figure 10. Composite pavement cross sections: low-modulus jointed concrete base.



* Filter layer located between drainage layer and underlying layers, if required.

Figure 11. Composite pavement cross sections: low-modulus CRC base.



* Filter layer located between drainage and underlying layers, if required.

In accomplishing this, some extrapolation beyond the normal range of experience was required. Because of this extrapolation, there are certain aspects of the design procedure that may not be as reliable as others. Other factors must remain under the direct decision of the engineer, and no matter how well proved the concepts are, the engineer must provide the correct input and quality control or the pavement's performance will be less than desired.

Any pavement not properly constructed will fail to provide the high level of performance required for the minimum 20-year design. Proper inspection and control to correct material or construction deficiencies must be accomplished in the field. Other factors not well established in terms of performance and of critical concern to the design engineer include low-modulus PCC durability, environmental considerations, and pavement subsurface drainage. In this study, every attempt has been made to investigate available performance data and to synthesize a reasonable and coherent design procedure that considers the effects of changing environmental and material conditions.

In conclusion, the concepts and criteria presented here are recommended for use and study in future practical field implementation by experienced design engineers to increase the reliability of this procedure and to provide additional performance data for needed revisions.

ACKNOWLEDGMENT

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REFERENCES

1. H.L. Von Quintus, F.N. Finn, W.R. Hudson, and F.L. Roberts. Flexible and Composite Structures for Premium Pavements--Volume 1: Development of Design Procedure. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-81-154, Dec. 1979.
2. H.L. Von Quintus, F.N. Finn, W.R. Hudson, and F.L. Roberts. Flexible and Composite Structures for Premium Pavements--Volume 2: Design Manual. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-81-155, Dec. 1979.
3. H.R. Cedergren, J.A. Arman, and K.H. O'Brien. Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-73-14, Feb. 1973.
4. H.R. Cedergren. Drainage of Highway and Airfield Pavements. Wiley, New York, 1974.

5. Implemenation Package for a Drainage Blanket in Highway Pavement Systems. FHWA, U.S. Department of Transportation, May 1972.
6. M.Y. Shahin. Prediction of Low-Temperature and Thermal-Fatigue Cracking of Bituminous Pavements. Univ. of Texas, Austin, Ph.D. dissertation, Aug. 1972.
7. B.F. McCullough, A. Abou-Ayyash, W.R. Hudson, and J.P. Randall. Design of Continuously Reinforced Concrete Pavements for Highways. NCHRP, Project 1-15, 1975.
8. W.J. Kenis. Predicted Design Procedures--A Design Method for Flexible Pavements Using the VESYS Structural Subsystem. Proc., 4th International Conference on the Structural Design of Asphalt Pavements, Univ. of Michigan, Ann Arbor, Vol. 1, Aug. 1977.
9. J. Panak and H. Matlock. A Discrete-Element Method of Analysis for Orthogonal Slab and Grid Bridge Floor Systems. Center for Highway Research, Univ. of Texas, Austin, Res. Rept. 56-25, Aug. 1971.
10. H.J. Treybig, B.F. McCullough, P. Smith, and H. Von Quintus. Overlay Design and Reflection Cracking Analysis for Rigid Pavements--Volume 1: Development of New Design Criteria. FHWA, U.S. Department of Transportation, Final Rept. FHWA-RD-77-66, Jan. 1978.
11. E.S. Barber. Calculation of Maximum Pavement Temperatures from Weather Reports. HRB, Bull. 168, 1957, pp. 1-8.
12. R.E. Mills and R.F. Dawson. Fatigue of Concrete. HRB, Proc., Seventh Annual Meeting, 1928.
13. H.J. Treybig, M.F. McCullough, and W.R. Hudson. Tests of Existing Pavements and Synthesis of Design Methods. In CRC Airfield Pavements, U.S. Air Force Weapons Laboratory, Vol. 1, Dec. 1973.
14. W.R. Hudson and F.H. Scrivner. AASHO Road Test Principal Relationships--Performance with Stress, Rigid Pavements. HRB, Special Rept. 73, 1962, pp. 227-241.
15. A.S. Vesic and S.K. Saxena. Analysis of Structural Behavior of Road Test Rigid Pavements. NCHRP, Project 1-41(1), 1968.
16. The AASHO Road Test. HRB, Special Rept. 73, 1962.
17. P. Yimprasert and B.F. McCullough. Fatigue and Stress Analysis Concepts for Modifying the Rigid Pavement Design System. Center for Highway Research, Univ. of Texas, Austin, Res. Rept. 123-16, Jan. 1973.
18. K.D. Raithby and J.W. Galloway. Effects of Moisture Condition, Age, and Rate of Loading on Fatigue of Plain Concrete. In Abeles Symposium on Fatigue of Concrete, American Concrete Institute, Detroit, MI, ACI Publ. SP-41, 1974.
19. D.R. Snethen, L.D. Johnson, and D.M. Patrick. An Evaluation of Expedient Methodology for Identification of Potentially Expensive Soils. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-77-94, June 1977.
20. P.D. Thompson, D. Croney, and E.W.H. Currer. The Alconbury Hill Experiment and Its Relation to Flexible Pavement Design. Proc., 3rd International Conference on the Structural Design of Asphalt Pavements, London, England, Sept. 1972.
21. Fatigue-Coverage Concepts Applied to Concrete Airport Pavement Designs. Portland Cement Association, Skokie, IL, Tech. Paper, 1973.
22. M.I. Darter and E.J. Barenberg. Zero-Maintenance Pavement: Field Studies on the Performance Requirements and Capabilities of Conventional Pavement Systems. FHWA, U.S. Department of Transportation, Rept. FHWA-RD-76-105, April 1976.
23. N.W. Lister and C.K. Kennedy. A System for the Prediction of Pavement Life and Design of Pavement Strengthening. 4th International Conference on the Structural Design of Asphalt Pavements. Univ. of Michigan, Ann Arbor, Vol. 1, Aug. 1979.
24. Local Climatological Data: Annual Summaries for 1977, Parts 1 and 2. National Oceanic and Atmospheric Administration, National Climatic Center, Asheville, NC, 1977.
25. AASHTO Interim Guide for Design of Pavement Structure 1972. AASHTO, Washington, DC, 1974.
26. The AASHO Road Test: Report 5--Pavement Research. HRB, Special Rept. 61E, 1962.

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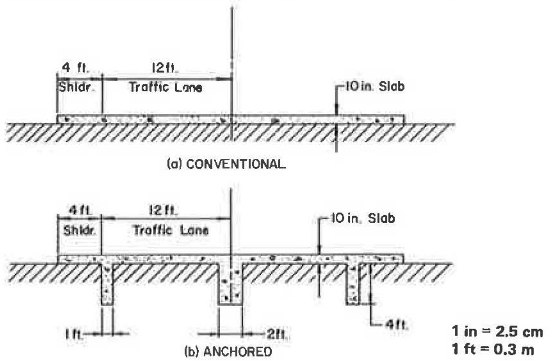
Model Study of Anchored Pavement

SURENDRA K. SAXENA AND S.G. MILISOPOULOS

A laboratory model study of an anchored pavement is described. The objective of the study was to investigate construction problems and development of specifications for a full-scale test. Also, the model tests could be and have been used to verify the analytical model. The model pavement involved 1/20-scale anchored and conventional slabs of similar dimensions and made of aluminum, a subgrade of known properties, a container tank for the whole setup, and loading and measuring equipment. In addition, one set of tests was performed by using the anchored slab in such a way that it is not in contact with the subgrade. The open space (void) between the slab and the subgrade simulates the worst conditions of no support caused by high moisture in the subgrade due to thaw or other actions. The model test results were compared with results from finite-element analysis. The investigations confirm that an anchored slab offers distinct advantages over a conventional slab; for example, the deflections are lower and uniform compared with those from a conventional slab, and stresses in the soil are reduced and distributed more widely by rigid anchors. The ANSYS computer program can analyze such a soil-structure system and incorporate the environmental and mechanical effects.

Serious concern about the maintenance costs and agonizing delays in repairs of highways in highly urban areas raised the question of the feasibility of designing and constructing minimum-maintenance pavements. As a result of research sponsored by the Federal Highway Administration, several structural concepts have been proposed (1) for minimum-maintenance performance. These include pile support, edge stiffening, thick cellular systems, waffle-type systems, modified conventional systems, and a flex-

Figure 1. Configuration of conventional and anchored pavements studied.



1 in = 2.5 cm
1 ft = 0.3 m

Figure 2. Views of ASL 62 in long.

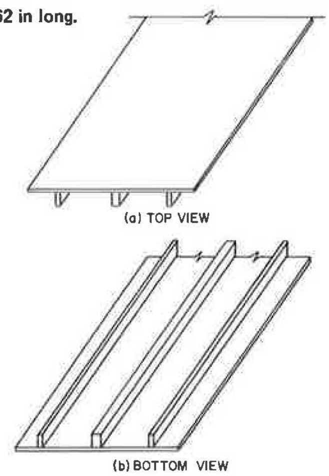


Figure 3. Plan view of testing tank and sampling tank.

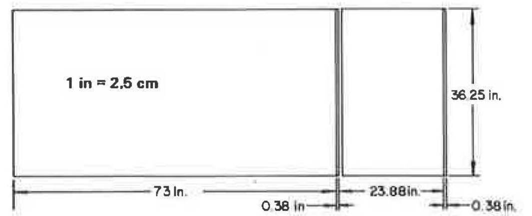
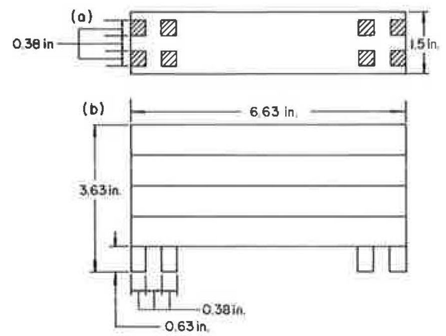
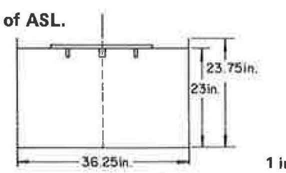


Figure 4. Loading platform: (a) plan view and (b) cross section.



1 in = 2.5 cm

Figure 5. Cross section of ASL.



1 in = 2.5 cm

ible "floating" V-shaped pavement. A limited study of various concepts showed an anchored pavement (Figure 1) to be promising because it uses a similar amount of structural material as current systems do, does not pose great construction difficulties, and may require little subgrade preparation. A laboratory model study of the anchored pavement is reported in this paper. The objective of the model study was twofold: first, to verify the results of the analytical model (computer program) and, second, to investigate construction problems and help to develop specifications for a full-scale test.

Experimental investigations of the structural behavior of rigid pavements have been made in the past at Arlington Experimental Farm in Virginia (2) and at Iowa State Engineering Experimental Station (3). Full-scale tests were performed at Schiphol Airport in Holland (4) and on Interstate 80 near Ottawa, Illinois, as part of the AASHO Road Test Program (this last test generated many studies). Model tests were performed under controlled conditions by Vesic and Saxena (5) to study the effect of stress due to load alone.

MODEL TESTS AND INSTRUMENTATION

The model tests for this investigation were 1/20 scale and involved anchored and conventional aluminum slabs of similar lengths, a subgrade of known properties, a container tank for the soil and for conducting the experiment, and loading and measuring equipment.

Slabs

Both the anchored and the conventional slabs were 62 in (157.5 cm) long, 21.63 in (55 cm) wide, and 0.5 in (1.27 cm) deep. For the anchored slab, there was an anchor near each edge and a central anchor. The two edge anchors measured 1.88 in (4.8 cm) deep and 0.625 in (1.6 cm) wide. The central anchor had the same depth but was 1.25 in (3.2 cm) wide. The lengths of the anchors and the slab were obviously the same. The anchors were attached to the slab by screws centered at 5 in (12.7 cm). Figure 2 shows the top and bottom views of the anchored pavement.

Subgrade and Container

The subgrade used was a soil made up of 42 percent kaolinite clay, 42 percent silica sand, and 16 percent water by weight. The subgrade was classified as silty clay that had a plasticity index of 16 and an optimum water content of about 8 percent. The soil was mixed with an over-the-optimum water content of 16 percent. A soil mixer and a compactor were used to prepare the subgrade. The silty-clay subgrade was mixed in 100-lb (45.36-kg) batches and then deposited in the tank, which was divided into two areas, testing and sampling (Figure 3). The testing area was 73 in (185.4 cm) deep. The sampling area was 27.88 in (70.8 cm) long; the other dimensions were the same. The subgrade was compacted in 2-in (5.08-cm) layers. For a uniform compaction, two passes of the compactor per layer were found to be sufficient. Before another layer was placed, the top 0.5 in (1.27 cm) of the previous layer was raked to ensure a proper bond. Every effort was made to have uniform compaction in all layers. The uppermost layer was leveled with

Figure 6. Positions of load, dial gages, and surface deflections.

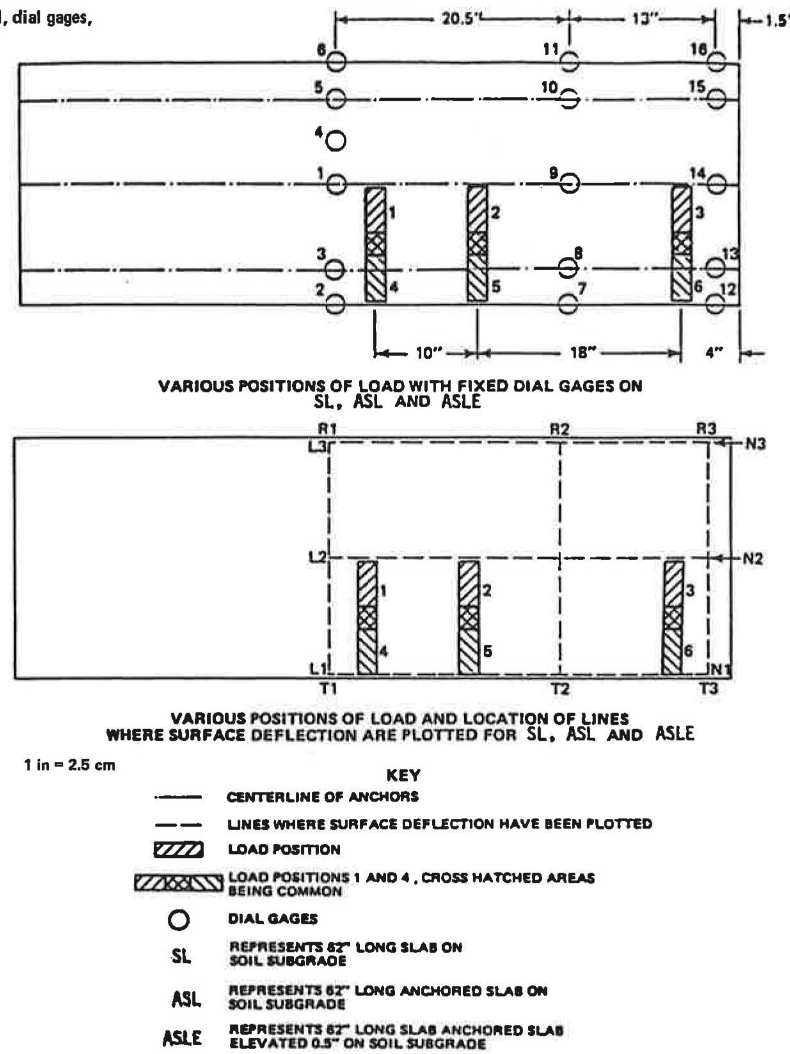


Figure 7. Positions of dial gages on ASL.

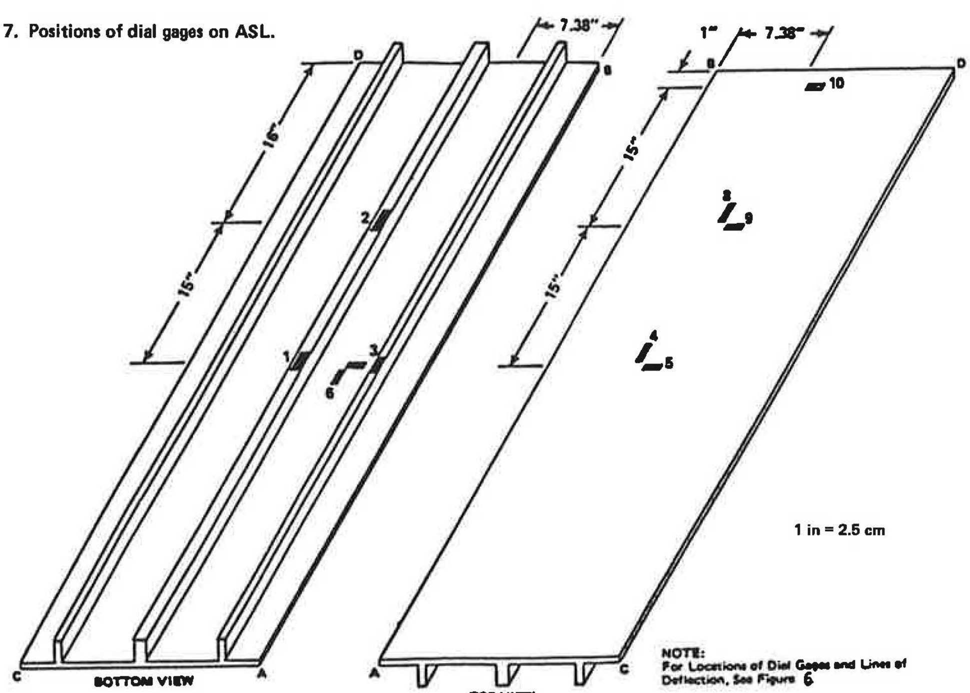


Figure 8. Load at center of ASL (traffic lane).

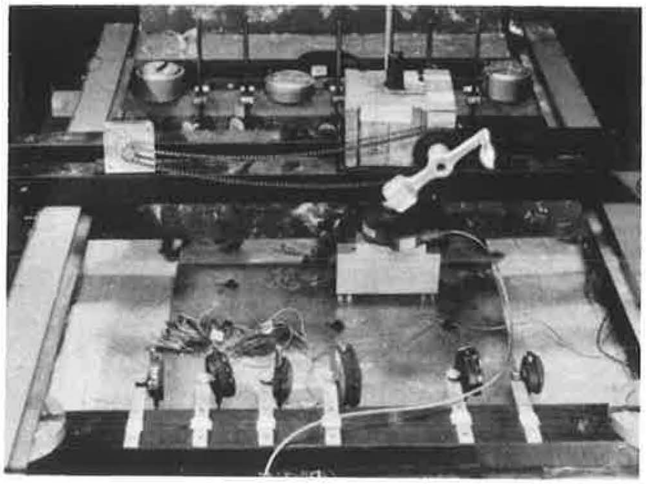
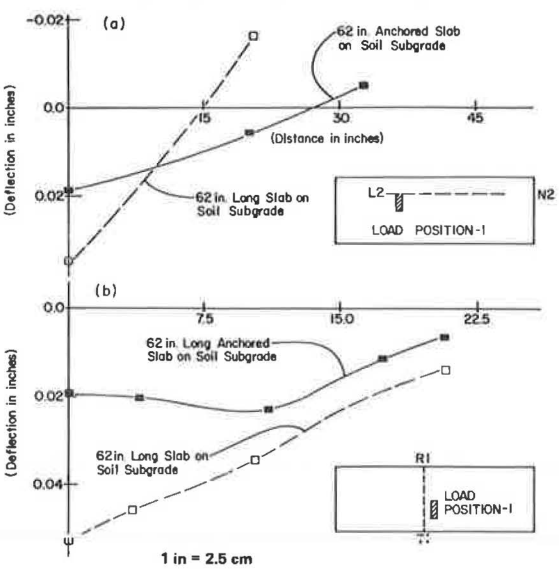


Figure 9. (a) Longitudinal and (b) transverse surface deflections along lines L2-N2 and T1-R1, load position 1, for ASL and SL.



precision to ensure proper contact between the slab and the subgrade. When the anchored system was used, anchors were placed before the leveling of the upper layer; then, after leveling, the slab was screwed to them. To avoid disturbing the soil, this method was considered better than welding the anchors to the plate. Crossbars were used to keep the anchors in position during compaction; the crossbars were removed later.

The properties of the subgrade were measured by performing many tests on undisturbed specimens obtained from the sampling area and by a plate-load test performed in situ. An odometer-type K_o-test provided an overconsolidation ratio of 30 and a value of K_o equal to 2.6. The observed value of K_o is in agreement with that in the published literature for overconsolidated clays (6) and compacted clays (7). The consolidated undrained triaxial tests provided a value of cohesion intercept $\bar{c}$ = 200 lbf/in² (9.58 kN/m²) and angle of interval friction $\bar{\phi}$ = 18°. The plate-load tests provided a secant mold of 850 lbf/in² (5865 kN/m²). The Poisson ratio adopted was ν = 0.4. The mold of elasticity of the aluminum slab was 10.5×10^6 lbf/in² (72.45×10^6 kN/m²).

Loading and Measuring Equipment

The loading platform (Figure 4) was designed to represent a 1/20-scale model of two rear-axle trucks that have four tires per axle and a maximum capacity of 18 000 lb (8165 kg) per axle. The 18 000-lb/axle load on a 1/20 scale was equal to a 45-lb (20.4-kg) load; however, heavier loads were used within the elastic range to investigate the response of the slab-subgrade system. For load application, a manual jack that had an attached load cell was used. A strain-indicator unit and balance unit were used to measure the strains of 10 foil strain gages fixed at various locations on the slab. The following notation was used for the test series.

1. Anchored slab, 62 in, laid on soil subgrade--ASL;
2. Conventional slab, same length, on soil subgrade--SL; and
3. Anchored slab, same length, elevated by 0.5 in from soil subgrade--ASLE.

TEST DETAILS

Anchored Slab on Soil Subgrade (ASL)

A cross-sectional view of the ASL is shown in Figure 5. The positions of the dial gages for measuring surface deflections of the anchored slab are shown in Figure 6. The 16 numbered points at which the deflections were measured were used to draw transverse and longitudinal deflections along sections T1-R1, T2-R2, and T3-R3 and along sections L1-N1, L2-N2, and L3-N3, respectively (Figure 6). Ten dial gages were placed on the slab to measure strain and deduce the stress and bending movement. Five dial gages were placed on the top surface of the slab, two on the bottom surface, and the other three on the anchors (Figure 7).

A line preloading was initially used at the center, quarter distances, and edges of the anchored slab. The preloading was considered necessary to ensure contact between the slab and the soil and also to bring the soil within elastic range. It was found that after about five loading and unloading cycles, the soil was within elastic range. As shown in Figure 6, six load positions were used to investigate the response of a continuous pavement and that of a pavement at a joint that has zero transfer load. The load was applied in increments of 250 lb (113.4 kg). Although the applied maximum load was 1500 lb (680.4 kg), the plots in the paper only show deflection for 250-lb, 500-lb (227-kg), and 750-lb (340-kg) loads. Figure 8 shows one of the various loading configurations, and Figures 9 and 10 show longitudinal and transverse deflections at centerlines (L2-N2 and T1-R1) for load at positions 1 and 3 only.

Conventional Slab on Subgrade (SL)

Deflections were measured at the same 16 locations shown in Figure 6. Six dial gages were attached to the top of the slab and four were attached to the bottom. After preloading in the center, at the left, and at the right intermediate sections, regular loading was applied. A maximum load of 750 lb that had a load increment of 250 lb was used at six loading positions (Figure 6). The deflections are shown with the anchored-slab deflections for only two loading positions.

Figure 10. (a) Longitudinal and (b) transverse surface deflections along lines L2-N2 and T1-R1, load position 3, for ASL and SL.

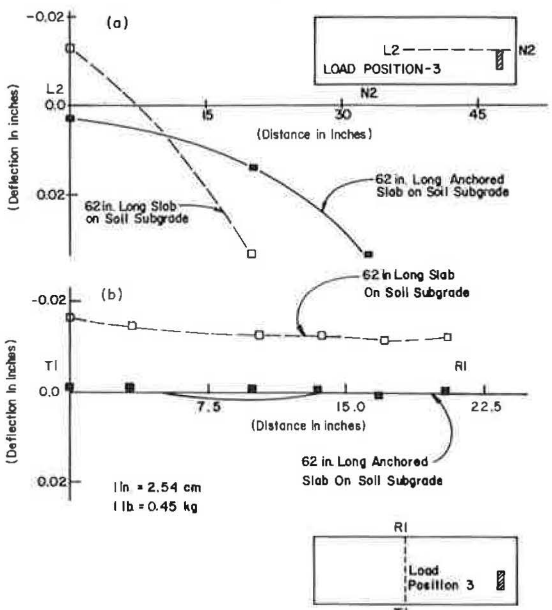


Figure 11. Cross section of ASLE.

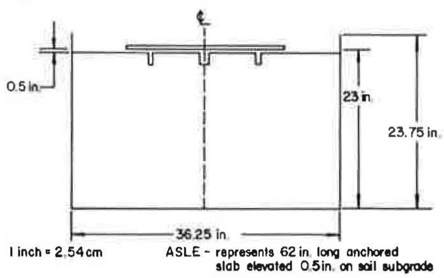
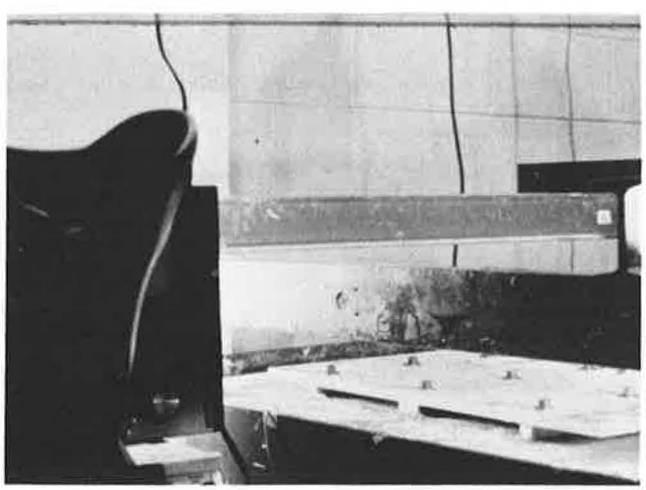


Figure 12. ASLE experimental set-up.



Elevated Anchored Slab (ASLE)

The procedure was similar to that for previous experiments. Before the slab was screwed to the anchors, a 0.5-in (1.27-cm) space was left between the slab and the subgrade. The space simulates the worst conditions of no-support capability caused by high moisture in the subgrade due to thaw or other action. Figure 11 shows a cross section of the model, and Figure 12 is a view of the experimental set-up. Preloading was done at the center, at the quarter distance on the left, at the quarter distance on the right, and near the edge on the left and the right to bring the soil into elastic range. The preloading was done symmetrically to avoid any possible lifting of the anchored slab. After preloading, the slab was loaded at six positions as described before. The maximum load applied was 300 lb (136 kg) in increments of 100 lb (45.36 kg). The load being applied to the subgrade at concentrated points had to be low enough not to cause plastic (or bearing-capacity-type) failure. Figures 13 and 14 show longitudinal and transverse deflections for the middle section only for load positions 1 and 3.

TEST RESULTS

The surface deflections of the anchored slab in the longitudinal direction and the central transverse section for the central loading position (position 1) are about one-third of those obtained for a conventional slab. At the quarter distance from the edge and near the edge, the conventional slab exhibited significant uplifting, whereas the anchored slab had almost no uplift or insignificant uplift only at the edge. Similar trends--that is, differential magnitudes of the order of one-third--were observed for the edge loading (position 3), as shown in Figure 10. The uplifting of the center was very pronounced for the conventional slab but insignificant for the anchored slab (Figures 9a and 10b). The anchored slab was also compared with the elevated-slab system. The surface deflections beneath the load for position 1 of the anchored slab are about one-third to two-thirds of the deflections for the elevated slab. No significant uplifting was observed in the elevated slab. The experiments indicate clearly that for a load of 250 lb only, the deflections of the anchored slab in full contact (Figures 13 and 14) and those of the elevated pavement were found to be very similar. This shows that a considerable amount of load is carried by anchors to the soil beneath.

ANALYTICAL SOLUTION

The analytical solution uses the options available in computer program ANSYS. In this section a comparison of anchored pavement with the conventional slabs and with the elevated anchored slab (which represents loss of contact below the slab) is presented.

Eight-node brick elements were used in the finite-element analysis (three-dimensional analysis). In case of the ASL and the conventional slab (SL), interface elements of infinitesimal length were used between the slab and the subgrade soil. Thus the effect of the uplifting, when the slab loses contact with the subgrade soil, was taken into account.

SIMPLY SUPPORTED SLABS

To verify the computer program, analyses of a simply supported anchored slab (SASL) and a simply supported conventional slab (SSL) were performed first and compared with experimental results. In the experiment, a line load of 2.95 lbf/in^2 (20.36 kN/m^2) was applied 1 in from the transverse centerline. The longitudinal deflections at the centerline are plotted in Figure 15, which shows the experimental deflection, the deflection obtained by

Figure 13. (a) Longitudinal and (b) transverse surface deflections along lines L2-N2 and T1-R1, load position 1, for ASL and ASLE.

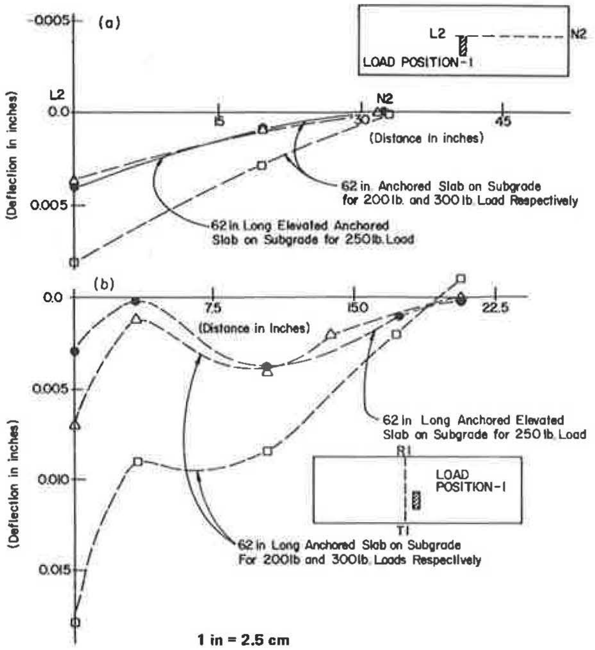


Figure 14. (a) Longitudinal and (b) transverse surface deflections along lines L2-N2 and T1-R1, load position 3, for ASL and ASLE.

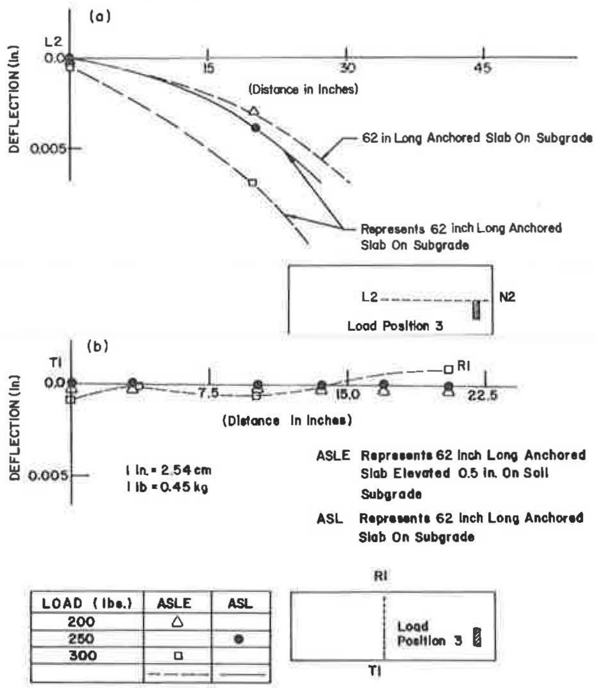


Figure 15. Longitudinal deflections at center for comparison of experimental, finite-element, and beam-theory models.

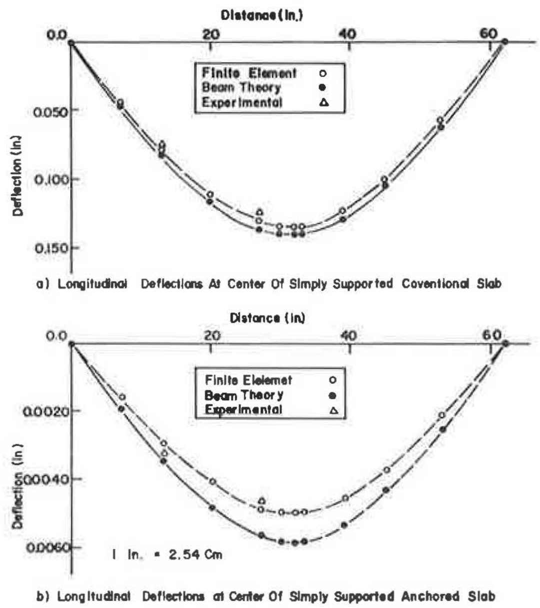


Figure 16. Longitudinal deflections at center for comparison of finite-element models of SSL and SASL.

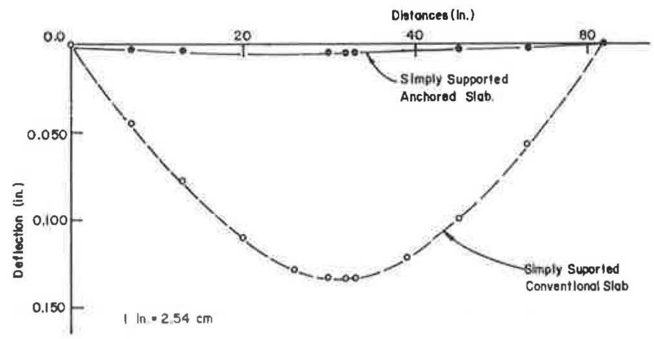
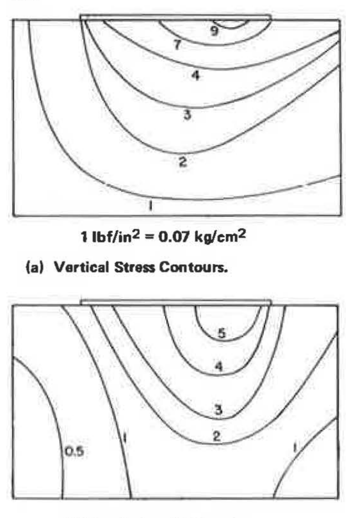


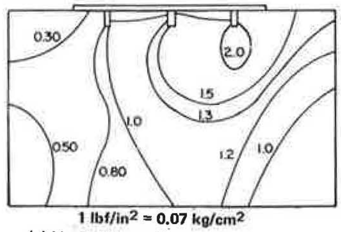
Figure 17. Vertical stress and strain contours for 750-lb load, position 1, for SL.



(b) Vertical Strain Contours.

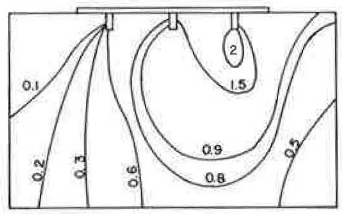
the finite-element method, and that obtained by the beam theory. The difference between the results from the finite-element method and the experimental values is about 7 percent for the SASL and about 4.5 percent for the SSL. The difference between the beam-theory approach and the experimental work is about 8.5 percent for both cases. Figure 16 shows the longitudinal deflections at the centerline of the SASL and the SSL by using the finite-element method. The intent is to demonstrate clearly the effect of the additional rigidity of the anchored

Figure 18. Vertical stress and strain contours for 750-lb load, position 1, for ASL.



$1\ lbf/in^2 = 0.07\ kg/cm^2$

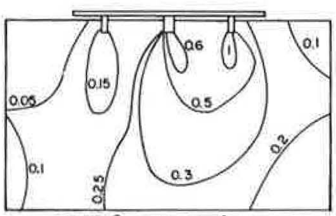
(a) Vertical Stress Contours.



1 Micro in./in. = 0.001 cm/cm

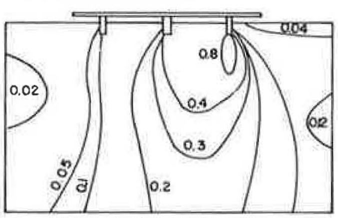
(b) Vertical Strain Contours.

Figure 19. Vertical stress and strain contours for 200-lb load, position 1, for ASLE.



$1\ lbf/in^2 = 0.07\ kg/cm^2$

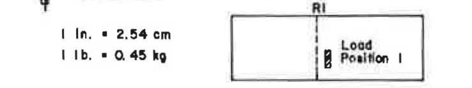
(a) Vertical Stress Contours.



1 Micro in./in. = 0.001 cm/cm

(b) Vertical Strain Contours.

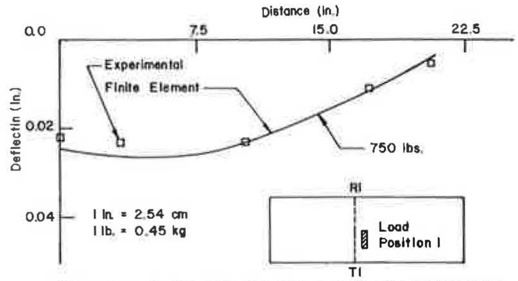
Figure 20. Transverse surface deflections along line T1-R1, load position 1, for SL.



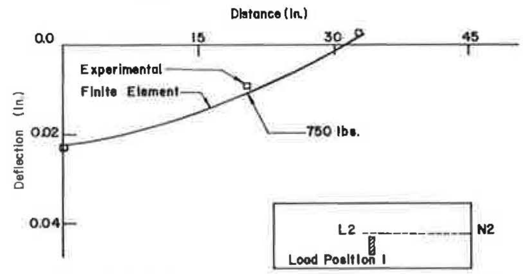
slab compared with that of the conventional slab.

The finite-element models for the ASL, the SL, and the ASLE are described by Saxena, Hedberg, and Ladd (6). For analytical investigation, a 750-lb load was applied at position 1 (see Figure 6) on the ASL and the SL, whereas a load of only 200 lb (90.72 kg) was applied at the same position for the ASLE. The vertical stress and strain contours only are presented for the three cases in Figures 17, 18, and

Figure 21. Transverse and longitudinal surface deflections along lines T1-R1 and L2-N2, load position 1, for ASL.



a) Transverse Surface Deflection Along Line T1-R1, Load Position 1 Anchored Slab.



b) Longitudinal Surface Deflection Along Line L2-N2, Load Position 1 Anchore Slab.

19. In the case of the SL, a maximum stress of 9 lbf/in^2 (62.1 kN/m^2) and a maximum strain of 0.005 in/in (0.013 cm/cm) underneath the load was observed. The analytical results also indicate a maximum deflection of 0.1 in (2.54 cm) under the load, and more than half of the slab loses contact with the subgrade with maximum uplift of 0.012 in (0.30 mm) (1).

For the ASL, a maximum stress of only 2 lbf/in^2 (13.8 kN/m^2) and a maximum strain of 0.002 in/in (0.030 cm/cm) underneath the right anchor was observed. The analytical results indicate a maximum deflection of 0.031 in (0.8 mm) in the right anchor (closest to the load) and almost no serious uplift of the anchors except at the left and right edges, which experienced a maximum uplift of 0.004 in (0.1 mm).

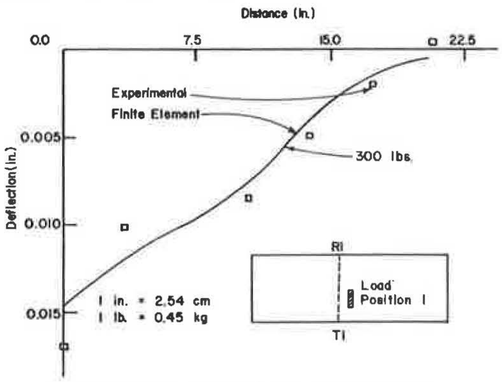
Figure 19 shows the stress and strain contours for the ASLE. A maximum stress of 1 lbf/in^2 (6.9 kN/m^2) and a maximum strain of 0.0008 in/in (0.0021 cm/cm) underneath the right anchor near the load is observed. The analytical investigations indicated a maximum deflection of 0.011 in (0.28 mm) at the right edge of the centerline near the load and no serious uplift. Only the far left edge is lifted; the maximum uplift is 0.001 in (0.03 mm). the central anchors carry 23 percent less load than the right anchors do, whereas the left anchors carry 49 percent less load than the right anchors do.

COMPARISON OF EXPERIMENTAL AND ANALYTICAL SOLUTIONS

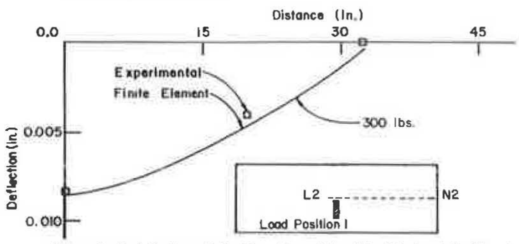
A comparison of the analytical solution with the experimental results is shown in Figures 20, 21, and 22. Figure 20 shows the deflection of the SL along the transverse centerline T1-R1 for a 750-lb load at position 1. The difference is about 20 percent. The experimental program on the SL was conducted after the loading for the ASL and the ASLE had been completed. A number of loadings and unloadings do strengthen the soil, and this is probably one of the major factors that contributes to the difference between the experimental and the analytical values.

In Figure 21, the transverse and longitudinal centerline surface deflections of the finite-element

Figure 22. Transverse and longitudinal surface deflections along lines T1-R1 and L2-N2, load position 1, for ASLE.



a) Transverse Surface Deflection Along Line TI-RI, Load Position I Elevated Anchored Slab.



a) Longitudinal Surface Deflection Along Line L2-N2, Load Position I Elevated Anchored Slab.

model of the ASL for a 750-lb load are plotted as well as the experimentally observed points. The difference is about 11 percent. Figure 22 presents the transverse and longitudinal deflections and the experimental points for the ASLE for a 300-lb load at position 1. Although the longitudinal deflections show remarkable agreement, there is a difference of about 18 percent between observed and analytical values for the transverse deflections.

Although every effort was made to maintain the uniformity of the subgrade, human factors do cause nonuniformities. Keeping these factors in mind, it may be remarked that, in general, the trends of observed deflection of the model tests agree well with the analytical (finite-element) results. Although the paper presents results of only a few of the many experiments conducted, the inferences are based on a study of all experimental results.

CONCLUSIONS

The anchored slab offers two distinct advantages over the conventional slab. First, deflections are lower and more uniform. Second, stress in the soil is lower and distributed more widely by the rigid anchors. A significant portion of the pavement-distress mechanism arises from the subgrade, in which the soil is under greater confining stress (and as a result is stronger), and when moisture and temperature fluctuations are not acute, subgrade-related failure is less likely to occur.

REFERENCES

1. S.K. Saxena, J.W.H. Wang, J.J. Udwari, and W.J. Rosenkranz. Unique Concepts and Systems for Zero-Maintenance Pavements. FHWA, U.S. Department of Transportation, Rept. FHWA/RD-77-76, July 1977.
2. L.W. Teller and E.C. Sutherland. The Structural Design of Concrete Pavements. Public Roads, Vol. 16, Nos. 8 and 9, 1935, pp. 145-158, 169-197; Vol. 16, No. 10, 1935, pp. 201, 221; Vol. 23, No. 8, 1943, pp. 167-212.
3. M.G. Spangler. Stresses in Concrete Pavement Slabs. Proc., HRB, Vol. 15, 1935, pp. 122-146.
4. C. Vander Veen. Loading Tests on Concrete Slabs at Schiphol Airport. Proc., 3rd International Conference on Soil Mechanics and Foundation Engineering, Switzerland, 1953, pp. 133-141.
5. S.K. Saxena and A.S. Vesic. Experimental Study of Slabs Resting on a Silty Clay Subgrade. Presented at the 57th Annual Meeting, HRB, 1978.
6. S.K. Saxena, J. Hedberg, and C.C. Ladd. Geotechnical Properties of Varved Clays of Hackensack Meadow, N.J., U.S.A. ASTM Geotechnical Testing Journal, Vol. 1, No. 3, Sept. 1978.
7. T.H. Wu. Retaining Walls. In Foundation Engineering Handbook, Van Nostrand Reinhold, New York, 1975.
8. S.K. Saxena, W.J. Rosenkranz, and S.G. Militsopoulos. New Structural Systems for Zero-Maintenance Pavements, Volume 1--Analytical and Experimental Studies of an Anchored Pavement: A Candidate Zero-Maintenance Pavement. FHWA, U.S. Department of Transportation, Rept. FHWA/RD-80-026, May 1980.
9. M.G. Spangler and F.E. Lightburn. Stresses in Concrete Pavement Slabs. Proc., HRB, Vol. 17, 1937, pp. 215-234.

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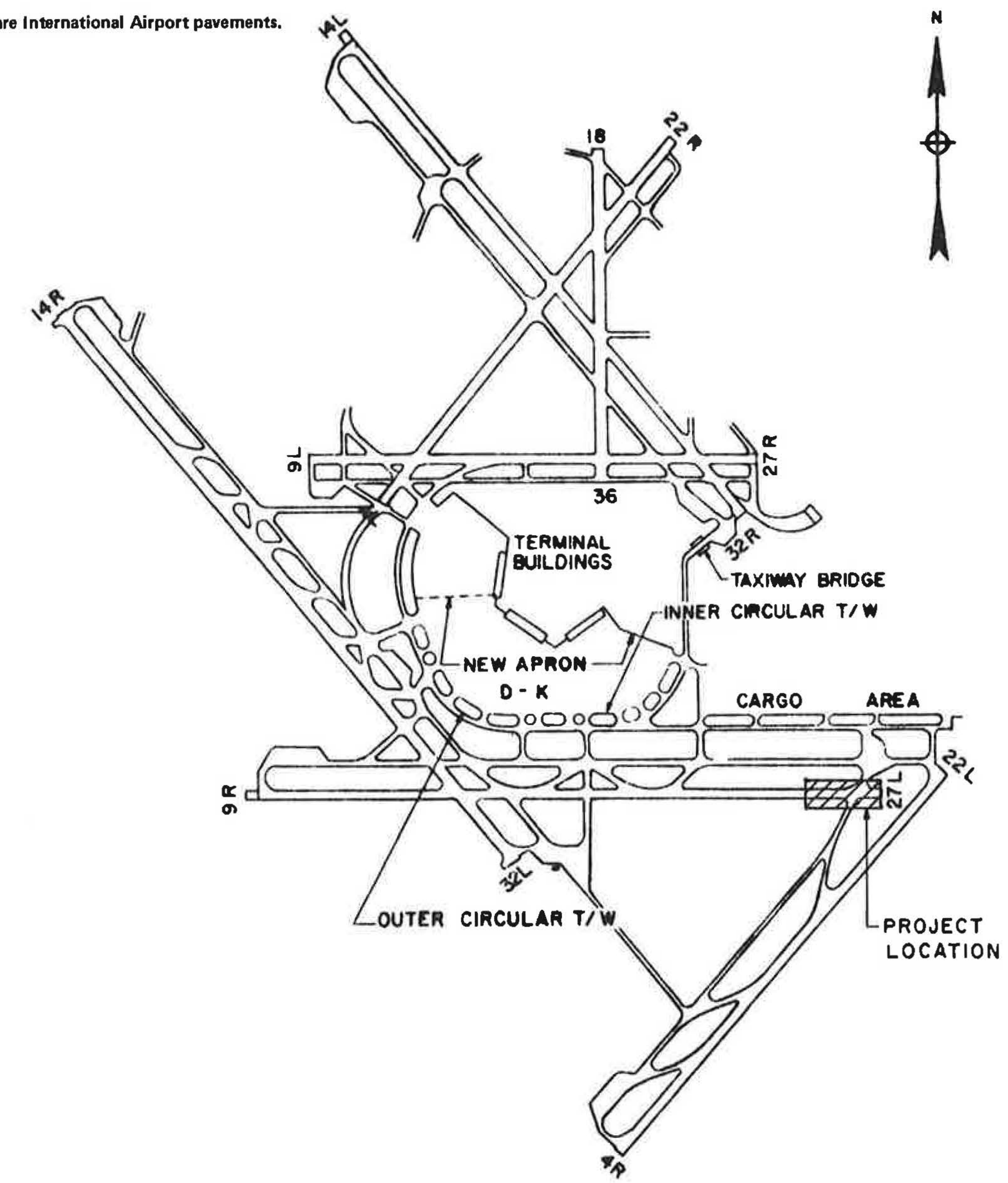
Prestressed Concrete Overlay at O'Hare International Airport: In-Service Evaluation

DONALD M. ARNTZEN

A 240-m (800-ft) prestressed concrete overlay was placed on the 27L end of runway 9R-27L at Chicago O'Hare International Airport. The overlay consisted of two 120 x 46-m (400 x 150-ft) sections 20 or 23 cm (8 or 9 in) thick. The pavement was posttensioned by using a fully bonded bar system. Conventional paving and tensioning equipment was used, and the cost and time of construction were comparable with those of conventional paving systems of portland cement concrete.

At 10:00 a.m., Sunday, August 24, 1980, runway 27L at Chicago O'Hare International Airport was reopened for traffic after completion of a prestressed concrete overlay. This overlay of the threshold 240 m (800 ft) long was another step in the ongoing program to rehabilitate and upgrade the airfield pavement system.

Figure 1. Layout of Chicago O'Hare International Airport pavements.



BACKGROUND

Chicago O'Hare International Airport has six primary runways and one secondary runway, which amounts to 814 954 m² (974 825 yd²) of pavement. The taxiway system amounts to 1 104 930 m² (1 321 687 yd²) and the apron system amounts to 633 278 m² (757 510 yd²). This totals 2 553 162 m² (3 054 022 yd²) of airfield pavement, which equals 349 km (217 miles) of pavement 7.3 m (24 ft) wide. The airport pavement layout is shown in Figure 1.

The original runways 4L-22R, 9L-27R, 14L-32R, and 18-36 built in 1942 consisted of 18-cm (7-in) portland cement concrete (PCC) on 30.5 cm (12 in) of gravel. Subsequent construction of runways, taxiways, and aprons has included 30.5-cm PCC and continuously reinforced concrete pavement (CRCP), 38.4-cm (15-in) PCC, 35.6-cm (14-in) CRCP, 45.7-cm (18-in) PCC, and 53.3-cm (21-in) PCC.

In 1974, a systematic program of pavement rehabilitation began with the bituminous concrete overlay of runways 9R-27L, 14R-32L, and 14L-32R. In subsequent years, runways 9L-27R and 4L-22R along with numerous taxiways were also overlaid with bituminous concrete. The runway overlays were expected to perform for five years before additional surfacing was required; consequently, a staged program was adopted whereby the total overlay was not installed initially. By now, 53 percent of the runways have been overlaid with bituminous concrete.

ECONOMICS

The 53.3-cm PCC on 15 cm (6 in) of bituminous asphaltic mixture, adopted as a standard for O'Hare, was selected after completion of the Airfield Pavement Demonstration-Validation Study (1). Although the 53.3-cm standard section is the most economical, the cost of removal and replacement has risen to \$110/yd². Because of grade differentials, it is necessary to remove the existing pavement, which contributes to the cost and the length of time of construction. The cost of delays has also risen dramatically, and a Delay Task Force Study (2) was completed in July 1976 for Chicago O'Hare International Airport. This report indicates that the annual cost of aircraft delays that could occur without optimized runway use is $27.6 million. This does not infer that all runway construction will affect operations equally; however, even occasional disruptions are costly. Because of these delay costs, airline representatives are advocating nighttime construction.

PRESTRESSED CONCRETE PAVEMENT

At the 1979 International Air Transportation Conference in New Orleans, Louisiana, Richard Heinen presented a paper on prestressed concrete pavement design and described the various installations in Europe and South America (3). At that time, 3 261 300 m^2 (3 900 050 yd^2) had been placed since 1956, and it was reported that these pavements were performing satisfactorily. The thickness of the prestressed pavements varied from 14 to 18 cm (5.5-7 in), and they were used by all types of aircraft, including the Boeing 747.

Members of the American Society of Civil Engineers' Airfield Pavement Committee inspected the prestressed pavements at Amsterdam's Schipol Airport, the Cologne-Wahn Airport at Cologne, and the Manching NATO/German Air Force Base near Munich and concluded that an in-service evaluation in the United States was warranted. Of particular interest was the possibility of using prestressed pavement as an overlay, which would provide a maintenance-free pavement that would have minimal grade changes. This concept was approved by the city of Chicago and the Federal Aviation Administration, and the project to rehabilitate the 27L end of runway 9R-27L was selected for the evaluation.

DESIGN CONSIDERATIONS

The prestressed concrete overlay was to be patterned after the successful installations inspected in Europe. Major structural failures in the existing CRCP such as punchouts or base failures would be repaired. Normal transverse cracking or longitudinal construction joints would not be reinforced. The pavement would be designed for a 20-year service life by using the finite-element model with edge loading for maximum stress. Verification of the design would be determined by using instrumentation and actual aircraft loading.

DESIGN CRITERIA AND RUNWAY DATA

The installation was not to be experimental but was to be patterned after the systems installed in Europe and South America. Consequently, a posttensioned, fully bonded bar system was used. The design aircraft was to be the Boeing 747, which has a wheel loading of 22 197 kg (49 000 lb). The spacing of the main gear wheel is 147 cm (58 in) longitudinally and 112 cm (44 in) transversely. The reaction of the modulus of the base course on the surface of the 31-cm CRCP was assumed to be 51.09 kN/cm^3 (700 lbf/in^3). The 14-day compressive stress of the concrete was specified to be 34 500 kN/m^2 (5000 lbf/in^2). The concrete was to be air-entrained and the surface grooved. Centerline lights were to be installed, and provisions for future touchdown lights were to be made.

Runway 9R-27L is 3065 m (10 140 ft) long and 46 m (150 ft) wide and has 8-m (25-ft) shoulders. It is one of the preferential runways in the optimized runway-use plans and consequently is subject to heavy traffic. Annual operations at O'Hare have exceeded 700 000 arrivals and departures. In addition to extensive use for arrivals and departures, the runway threshold is used by aircraft that taxi from runway 4R-22L. This cross traffic is one of the causes of the failure of the threshold.

PAVEMENT DESIGN

The pavements previously installed in Europe were 14-18 cm thick. The 18-cm pavements had been subjected to B747 loadings in international operations; however, these loads were not so frequent as those at O'Hare. These pavements were stressed at each end, which required a gap for jacking filled by using a gap slab. Also, each transverse joint was laid on a sleeper slab.

The design analysis for 27L by using sector analysis and the finite-element model resulted in a 20-cm (8-in) pavement thickened to 23 cm (9 in) at the longitudinal joint adjacent to the taxiways. It was decided that if prestressed concrete was to be effective as an overlay, it would be desirable to eliminate the sleeper slab and the gap slab. Consequently, two thicknesses were selected as shown in Figure 2, and a sleeper slab was used only at the joint between the 20-cm and the 23-cm slabs. The east and west ends were poured directly on the existing concrete, which had a PCC leveling course 5 cm (2 in) thick by 61 cm (24 in) wide bonded to it. The balance of the pavement was overlaid with an asphaltic concrete leveling course 2.5-5 cm (1-2 in) thick. Stressing of the tendons both longitudinally and transversely was to be from one end only, i.e., the east, west, or south end of the completed overlay.

CONSTRUCTION

The successful low bidder was Milburn Brothers, Inc., of Mt. Prospect, Illinois, and construction began on June 24, 1980. After the sleeper slab had been placed, major failures in the 30-cm CRCP had been repaired, and the asphaltic leveling course had been placed, standard metal forms lined with planking were installed. Two layers of polyethylene were laid on top of the asphalt, and then the conduits for the tendons, the light bases, and the electrical conduits were installed. The 2.5-cm longitudinal tendons were installed, and paving of the 8-m lanes proceeded by using a conventional paving train except that internal vibrators were not permitted. The contractor elected to use a superplasticizer to facilitate placement around the conduits and light bases. Burlap curing for seven days was specified; in the 39°C (101°F) environment, this proved beneficial in cooling the concrete during curing. In addition to the high temperatures encountered, more than 12.7 cm (5 in) of rainfall was recorded in this period. Because this operation was an overlay, it caused only minor interruptions in use of the airport, and the contractor did not request a time extension.

Prestressing in the longitudinal direction was performed at 12 h, at 36 h, and when the concrete had achieved 80 percent of its design strength. Prestressing in the transverse direction was performed after all lanes had been poured. PCC transition beams 1.22 m (4 ft) wide doweled into the existing pavement were placed adjacent to the taxiways and the runway asphaltic concrete overlays.

After prestressing had been completed, an expansive grout was pumped into the conduits to fully bond the tendons. The tendons were deformed bars, and the grout specified required a pullout test of 6900 kN/m^2 (1000 lbf/in^2). The concrete was a 6.5-bag mix that included a limestone coarse aggregate [maximum size, 2 cm (0.75 in)], which was specified to decrease the potential of D-cracking. At other locations at O'Hare, conventional 38-cm PCC pavements have failed due to D-cracking. These pavements had gravel as a coarse aggregate.

Grooving of the pavement was accomplished by us-

Figure 2. Plan of prestressed concrete overlay on runway 27L.

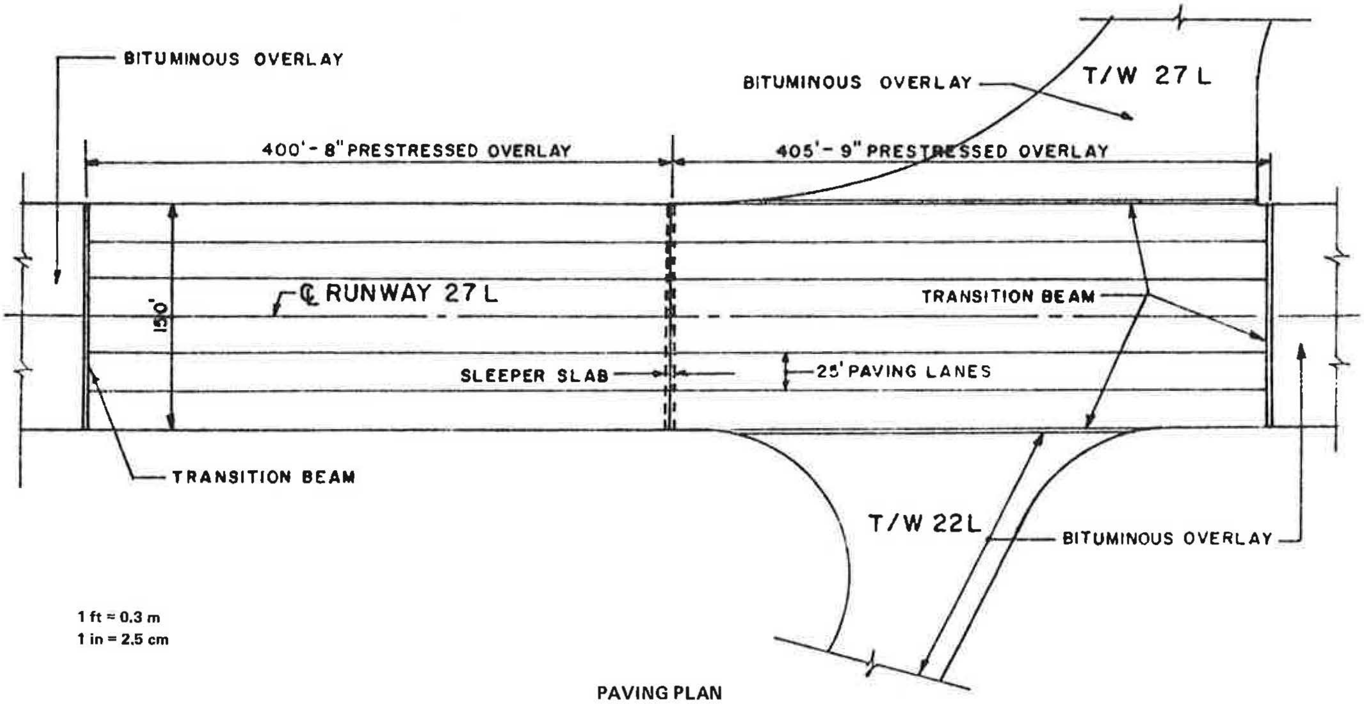


Figure 3. Paving train in operation.

Figure 4. Conduits and lighting fixtures in place.

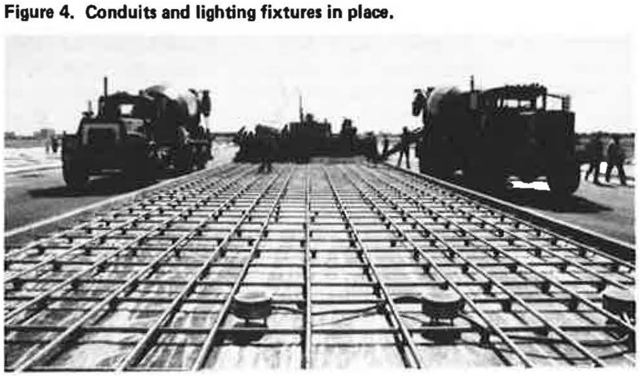
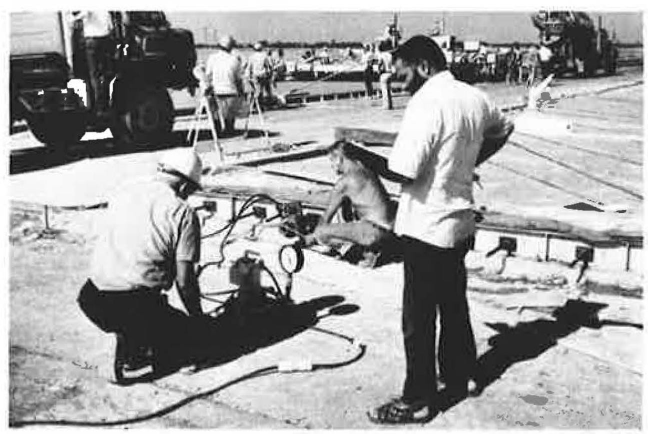


Figure 5. Tensioning longitudinal tendons.



ing a wet diamond-sawing process.

Test sections for deflection were installed at six locations. On Sunday, August 24, 1980, from 7 to 8 a.m., a United Airlines B 727-200 that weighed 59 362 kg (131 000 lb) was positioned over the test locations in the 23-cm and in the 20-cm overlays. Prior to the test, calculations had been made for various subgrade moduli so that field measurements could be interpreted as they were taken. The results obtained during the test indicated that the subgrade moduli were significantly in excess of those used in the design and that the edge that did not have a sleeper slab was more than adequate.

CONCLUSIONS

Based on the bids received and time of construction, it is apparent that prestressed concrete overlays are competitive with other PCC paving systems. From observations of the progress of the construction, the equipment used, and the lighting installation, there are no apparent problems associated with using standard paving trains or standard posttensioning equipment (Figures 3, 4, and 5). The load testing indicated that the system could span joints or cracks and that sleeper slabs were not required. Also, varying thicknesses of prestressed pavement could be placed.

Long-term observations will be necessary to document maintenance costs and possible design modifications to improve on the construction and performance of a prestressed overlay system.

It is strongly recommended that qualified, experienced personnel be employed in the design, construction, and quality assurance of prestressed concrete pavements.

REFERENCES

1. D.M. Arntzen, E.J. Barenberg, and R.J. Krause. Airfield Pavement Demonstration-Validation Study. Transportation Engineering Journal of ASCE, Vol. 106, No. TE6, Proc. paper 15813, Nov. 1980, pp. 721-730.
2. Delay Task Force Study, Volume 1: Executive Summary. Chicago O'Hare International Airport, Rept. FAA AGL-76-1-I, July 1976 (prepared through the joint efforts of FAA, Chicago Department of Aviation, and the airlines that serve O'Hare).
3. R. Heinen. Prestressed Concrete Airfield Pavements. International Air Transportation Conference, New Orleans, LA, May 1979, Proc., Air Transport Division of ASCE, Vol. 2, 1979, pp. 220-233.

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Bonded Portland Cement Concrete Resurfacing

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The experiences of the state of Iowa in developing and refining the process of resurfacing concrete pavements by using portland cement concrete (PCC) are described. The methods of evaluating the condition of the underlying pavement and determining the thickness of the resurfacing layer are discussed. Several projects that used PCC resurfacing to satisfy different roadway needs are described. Several methods of surface preparation, the methods of bonding, and the bond test results are included and discussed. It is concluded that bonding a layer of PCC 50-75 mm (2-3 in) thick to an existing concrete pavement is a viable alternative to bituminous resurfacing for the rehabilitation and restoration of concrete pavements.

Iowa has more than 12 000 miles of portland cement concrete (PCC) primary and Interstate highways, secondary or county roads, and city streets. Approximately 15 percent of these streets and highways have been in service more than 40 years and have had little or no surface maintenance and no additional wearing surface. Many, however (especially those that carry high volumes of traffic), need surface attention at this time. The serviceability (rideability) is approaching, and in some cases has arrived at, the point at which surface restoration or reconstruction is imminently needed.

Nationally and locally, the trend has shifted from building miles of new pavement to restoring and rehabilitating the existing pavement. This has been for the most part due to financial, environmental, and ecological restrictions.

Historically, the restoration process on PCC roads and streets has usually involved resurfacing by using bituminous materials to provide an acceptable riding surface. The bituminous-resurfacing process has provided city, county, and state government agencies with a viable method of extending the service life of PCC pavement at a considerably lower cost than that of reconstructing or replacing the facility.

Since 1976, this nation has been made aware that petroleum and products derived from petroleum are becoming more and more expensive. Further, and more important, is the forecast that this nation's supply of crude oil is quite limited and may be exhausted before the turn of the century. Thus, there is a stong emphasis on the search for substitute fuels, products, and methods that are not dependent on petroleum.

Various types of PCC overlays, which include plain, nominally reinforced, and continuously reinforced overlays, have been demonstrated on concrete pavements as well as a few cases on bituminous pavements. For example, since 1959, 13 different states, including Iowa (Greene County), have had projects that used continuously reinforced concrete overlays (1).

In 1973, a research project was conducted in Greene County that used 50- and 75-mm (2- and 3-in) thicknesses of fibrous reinforced concrete in various conditions of bonding: unbonded (two layers of polyethylene), partially bonded (wet interface), and bonded (dry cement broomed over wetted surface). Also, in the fall of 1954, PCC resurfacing was placed on US-34 in West Burlington, Iowa. This was reinforced by using steel mesh, and most of the project was bonded by using a nominal 6 mm (0.25 in) of cement-sand grout (2).

Although there is a variety of designs and construction procedures available, the projects mentioned above demonstrated the practicability of concrete for resurfacing in rehabilitating old concrete pavements. In previous attempts at full bonding of overlays, the information available indicates that complete bonding was not obtained.

A definite need existed for a high-strength, durable, skid-resistant, long-lasting, and economical resurfacing course for PCC pavements. Such a resurfacing course, completely bonded to the existing pavement, would provide additional support for the ever-increasing traffic loads and volumes on our roads and streets.

Iowa has had considerable success in the use of thin, bonded, dense concrete overlays used in the repair of deteriorated bridge decks (3). This process involves the removal of unsound concrete down to and around the top layer of steel reinforcement. The entire remaining area of the bridge-deck surface is removed to a nominal depth of 6 mm. This removal is most generally accomplished by using scarifying equipment.

The existing surface is scarified to remove road oils, linseed oil, etc., as well as the surface concrete that has the highest concentration of chloride ions from the interface. The entire surface is vigorously sandblasted and then air-blasted prior to the concrete-placement operation. The blasting is required to provide a clean dry surface to which a thin layer approximately 50 mm thick of dense, low-slump PCC is bonded. The bond is obtained by brushing on a grout of creamy consistency that consists of equal parts by weight of cement and sand immediately before the concrete placement.

By applying the same principles and methods learned from bridge-deck repair and resurfacing since 1965, it was felt that this system could provide a viable alternative to the bituminous resurfacing that has traditionally been used in the restoration and rehabilitation process on PCC pavements.

From the successful experience with bridge-deck repair and resurfacing in Iowa, it was expected that dense PCC could be placed and bonded to an existing concrete pavement. However, it was recognized that higher production, different equipment, and higher-slump concrete would have to be used to provide an economically viable process for large-volume projects.

A typical one-day bridge-deck resurfacing placement is 15-183 m (50-600 ft) long and 4-7 m (14-22 ft) wide and uses 19-mm (0.75-in) slump concrete on a prepared (ground or scarified) surface. This concrete is mixed in a small paddle mixer or a Concretmobile (a self-contained unit that produces concrete by using volumetric proportioning).

Obviously this rate of production would not be economical if a project 11-16 km (7-10 miles) long were to be resurfaced. Also, conventional paving equipment would require a higher-slump concrete for production and workability.

EVALUATION OF PAVEMENT FOR RESURFACING

In choosing a project location for the first attempt at thin bonded PCC resurfacing, the Iowa Department of Transportation (IDOT) looked for a project that would be considered typical and for which the traditional bituminous resurfacing would be the obvious corrective measure to take. In 1976, a section of concrete pavement of US-20 in Black Hawk County (northeast Iowa) was chosen for the first attempt at PCC resurfacing (4). This was a nonreinforced concrete pavement 200 mm (8 in) thick that was exhibiting deterioration in the form of D-cracking. There was considerable spalling of the transverse joints near the intersection with the centerline longitudinal joint. Some bituminous surface patch repair was already in existence.

The thickness for the first project was arrived at from previously mentioned experience gained from the bridge-deck repair system. It had been learned that machine placement of thicknesses less than 50 mm often resulted in tearing of the surface. A nominal 50-mm thickness is usually used in the bridge-deck repair and resurfacing system.

During the first project an attempt was made to duplicate, wherever possible, the already tried and tested system that was being used on bridge decks. The then-recent availability of the high-production scarifying machines in widths of approximately 2.4 m (8 ft) as well as the availability of superplasticizing admixtures provided the necessary items that enabled such a project to be attempted.

The self-propelled scarifying machine was able to break up the existing surface to a nominal depth of approximately 6 mm at a forward speed of approximately 6 m/min (20 ft/min). This provided a production rate that made the concept of applying the bridge-deck repair system to a highway project much more feasible.

The concrete mixture used on bridge-deck repair is a high-strength, low-slump (19-mm) mixture that cannot be placed by using existing conventional slipform paving equipment. The superplasticizing admixtures made possible the design of a concrete mixture that had a workability similar to concrete mixtures used in concrete paving but still maintained the very low water/cement ratios of traditional bridge-deck repair mixtures (the design water/cement ratio is 0.328 water to cement by weight).

This project clearly established that concrete that had a low water/cement ratio could be increased to conventional paving workabilities to allow its placement by using conventional slipform equipment in a thickness range of a nominal 50 mm. Also, it was demonstrated that mixtures of this type could be proportioned, mixed, and transported by means of transit mix trucks and that the inherent problems with the high slump loss that accompanies the use of superplasticizers could be and were overcome.

With the successful completion of the first demonstration project, an application that had a different requirement was investigated. This was a case of a relatively new concrete pavement 150 mm (6 in) thick that, because of changing traffic conditions after the road had been completed, was now considerably underdesigned.

After the road had been constructed in 1968, a commercial development that consisted of a grain terminal resulted in very heavy truck traffic on a road designed for normal secondary-road traffic. Thus the desire to increase the load-carrying capacity by bonding on a layer of concrete was the motivation for the second project.

In 1977, a 2.6-km (1.6 mile) research project was constructed in Clayton County, Iowa (5). An objective of this project was to determine the feasibility of proportioning and mixing a dense concrete mixture by using superplasticizing admixtures in a conventional central-mix batch plant. A second objective was to determine whether a conventional (without superplasticizers) paving mixture of conventional water/cement ratios, still in the nominal 50-mm thickness, would achieve an adequate bond and would adequately strengthen the section for the existing and anticipated traffic.

The successful completion of the project provided the necessary technique and process development to give IDOT confidence in the process. The procedure was considered a viable design consideration for rehabilitation and/or restoration of a deteriorated or deficient concrete pavement.

In 1978, still another condition was considered as a possibility for remedy by using a bonded PCC overlay. An existing four-lane divided facility had been widened by using PCC and later resurfaced by using asphaltic concrete. One particular 0.8-km (0.5-mile) stretch of this pavement was on a down grade in a 72-km/h (45-mph) speed zone and was carrying a considerable number of heavy trucks. At two traffic signals on this portion of pavement, the existing surface was exhibiting a washboarding or rippling effect caused by the braking action of the trucks. This section of roadway has been heater-planed several times to restore a smooth riding surface, and each time the result was short-lived and the roughness reoccurred.

A project was developed to remove the existing approximately 75 mm of asphaltic concrete, scarify the exposed concrete surface, and replace the removed asphalt resurfacing by using a bonded PCC overlay. This project was successfully constructed and completed in the spring of 1978 and is performing excellently.

With the experience described above, it was felt that the mixing, placement, surface preparation, bonding, etc., techniques were sufficiently developed and refined so that the process or method was ready to be used on a project in which it would be subjected to our highest traffic count and most severe loading. This would be on a section of our Interstate system.

It was felt that there were several things to learn from performance under traffic. A project site was chosen that consisted of two sections of PCC pavement. One section consisted of a continuously reinforced concrete (CRC) pavement 200 mm thick that abutted a 250-mm (10-in) mesh-dowel concrete pavement. Both pavements, which had been constructed in 1966 and 1965, were exhibiting deterioration in the form of D-cracking.

The CRC pavement was exhibiting a considerable amount of typical D-cracking deterioration along the longitudinal joint. There were several locations at which the secondary cracking was present in large areas, some as wide as a 3.6-m (12-ft) lane width and, on occasion, completely across the 7.3-m (24-ft) roadway. On the mesh-dowel section, the deterioration was present at the transverse joints that were spaced at 23.3 m (76.5 ft) as well as almost continuously along the centerline longitudinal joint.

Since this was to be Iowa's first experience with overlaying a CRC pavement, additional testing and evaluation were done in order to facilitate the design of the overlay or resurfacing thickness. Cores from the existing section were visually evaluated as well as tested for compressive strength. Due to its visible condition, it was obvious that this pavement needed some sort of rehabilitation and in certain areas was no longer a sound section for its full thickness.

The 75-mm thickness of PCC overlay for this project was influenced more by experience than by theoretical design. The performance of the previous projects and their traffic were considered. Also, it was known that there would be substantially more truck traffic on the Interstate, and this was considered in the decision to increase the thickness to more than that of previous overlays.

After the thickness decision had been made, the 75-mm design was evaluated by the normal pavement design procedure. Iowa uses the Portland Cement Association method, or rigid-pavement design (6). This method is based on fatigue of the concrete due to flexural stresses. Since it was expected that sufficient and adequate bond would be obtained to make the resurfacing or overlay act as a monolithic section, the thickness design was approached as that of a new monolithic pavement. It was determined that a new pavement section of plain PCC for this traffic under these conditions would need to be 267 mm (10.5 in) thick. It was felt that by scarifying existing pavement to a nominal depth of 6 mm, the existing pavement would result (as will be described later) in being approximately 190 mm (7.5 in) thick. Therefore, on the assumption that the remaining pavement was sound, an overlay 75 mm thick would provide a section considered to be sufficient to carry the design loading. It was recognized that, due to the D-cracking present, the remaining slab was not in a completely sound condition. For this reason, no consideration was given for the existence of the continuous reinforcement.

Another factor that contributed to the overall pavement structure and added conservatism to the design was that a drainage system was installed to remove water from directly under the pavement. Slotted polyethylene tubing 100 mm (4 in) in diameter was placed in a trench 250 mm wide located 610 mm (24 in) below the top of the existing pavement on each side. Outlets were provided at approximately 305-m (1000-ft) intervals. These drains were covered by using a designed porous backfill material. From previous experience with this type of drain installation, it was expected that stability of the subgrade would be substantially improved. This would directly affect the load-carrying capacity and longevity of a given pavement thickness.

The 75-mm bonded PCC overlay was placed on a 7.2-km (4.5-mile) section of I-80 in western Iowa in the summer of 1979. From all observations and evaluations to date, the resurfacing is performing as expected.

SURFACE PREPARATION

Since the beginning of the dense, low-slump-concrete bridge-deck repair system in Iowa in 1965, the importance of proper and complete surface preparation has been paramount. The success and long-term performance of any bonded overlay are directly related to a properly prepared surface.

In the bridge-deck repair system, the surface, as previously described, is scarified followed by a vigorous sandblasting. The scarifying is primarily to remove road oils and other surface contaminants from the interface and secondarily to provide a clean surface. The sandblasting operation is considered one that further cleans any oil drippings, spillages, etc., that might be left on the surface after scarification. The intent is to have a clean dry surface, free from any loose particles at the time of concrete placement. To remove any dust subsequent to sandblasting, the surface is cleaned by using an air blast just prior to the grouting and concrete-placement operation.

Through the various development stages of the thin-lift, bonded resurfacing technique, various other types of surface preparation were investigated and attempted. They were sandblasting alone, high-pressure water blasting alone, and a small amount of shot blasting. All these methods appear to be capable of providing a sufficiently clean surface with which to achieve sufficient and adequate bond by means of the grouting system that has been used to date. The only shortcoming of these techniques that we are aware of at this time is that in the single project in which high-pressure water blasting was used, the water blast was not capable of removing the painted traffic lines. It is important that paint, tire marks, etc., be removed in order to achieve a complete bond.

On the I-80 resurfacing project, the method of preparation chosen was that of scarification followed by sandblasting. The primary reason for this choice was the existence of a considerable amount of D-cracking on a large percentage of the roadway surface. It was felt that the scarifying effort would remove the partly fractured pieces, which would result in the sound surface needed for bonding the subsequent overlay. Specific locations of more-advanced D-cracking deterioration, such as along the centerline joint, were scarified to a nominal depth of 25 mm (1 in).

Due to the straight-line configuration of the mandrel, or cutting head, on the scarifying machine, to scarify a pavement that had a crown, generally depths in excess of 6 mm resulted. [The scarifying machine on this project used a mandrel that was approximately 3.6 m (12 ft) wide. Even by using a machine that was full-lane width, four passes were required in order to achieve complete coverage of the existing surface and not have to remove excessive amounts of pavement.]

It should be mentioned that any areas of deterioration such as blowups or punchouts that would normally be repaired by full-depth repairs for the traditional bituminous resurfacing were treated in a similar manner as part of the preparation for PCC resurfacing.

Pressure-relief joints have been constructed in Iowa's concrete-resurfacing projects to provide for the expansion and contraction forces caused by temperature changes. These have been approximately 100 mm (4 in) wide, spaced at approximately 305 m (1000 ft), and sawed full depth through the resurfacing roadway.

On the 1979 Interstate resurfacing project, the pressure-relief joints were sawed in the CRC pavement by using a large-diameter wheel saw prior to resurfacing. As soon as possible after resurfacing, the joint was sawed by using a diamond saw over the previously formed relief joint. It was then filled to full depth by using a polyurethane material.

At several relief-joint locations on the project, debonding of the adjacent resurfacing was detected prior to opening of the roadway. When the unbonded resurfacing had been removed, the old pavement appeared to be uniformly covered with bonding grout. It is presumed that there was some movement in the underlying pavement before the grout developed sufficient bond strength. This may have been caused by the increase in ambient temperature during placement in addition to that from the hydration of the resurfacing.

The unbonded overlay was removed and the areas were sandblasted, regrouted, and patched by using PCC. The repairs were successful. A change in pressure-relief joint design and construction will be considered for future projects.

BONDING PROCEDURE

In the first attempt at concrete resurfacing in Iowa, in 1954, a cement-sand grout of approximately 6 mm was placed (2). However, as indicated in the documentation of that project, the bonding layer not only was apparently excessively thick but was allowed to dry prior to the placement of the concrete. Thus, in all probability, it became a bond breaker rather than a bonding layer.

In the planning stages prior to Iowa's 1976 project of a bonded PCC resurfacing, a review of the research previously done on bonding was undertaken (7). This report of 10 years of performance of bonded resurfacings seemed to indicate no minimum bond strength, as measured in shear. It was determined that 1379 kPa (200 lbf/in²) is apparently adequate and that when such a bond is obtained, it will endure. The extensive amount of bond testing that has been done on the projects constructed to date in Iowa seems to confirm this.

Although small areas of delamination have been detected on various concrete-resurfacing and bridge-deck repair projects, cores have indicated the delamination to be below the interface in the vast majority of cases.

From core testing and observation to date (with the possible exception of the previously described experience at certain pressure-relief joints on CRC pavement), there has been no indication that bond has ever been lost once it had been attained.

The most critical factor that affects bond, in our opinion, is that the surface must be extremely clean and dry prior to the placement of the grout and subsequent placement of the concrete resurfacing. The virtues of a clean surface are readily apparent. The virtue of a dry interface results in some penetration of the grout into the existing or underlying surface. This creates an adequate and satisfactory bond.

Bond Testing

Much has been mentioned about the bond and bond testing. The testing jig used has been a shop-made shear collar, and the technique and test method were designed by and for IDOT. Basically, the jig consists of a two-part collar (Figure 1) that fits over a core 100 mm (4 in) in diameter; the junction of the two sliding parts is lined up over the bond line or interface between the underlying pavement and the resurfacing layer. The collar is then placed into a testing machine (Figure 2) and put in tension; the load required to shear off the resurfacing is then measured in kilopascals. This is recorded as the bond strength. The bond strengths that have been obtained to date from the various types of surface preparation are given in Table 1.

Grout Mixture and Application

The grout mixture that has been used on all bridge-deck repair projects as well as on all but the last resurfacing project has consisted of equal parts by weight of type 1 portland cement and sand that has enough water to make a creamy consistency. Care is

Figure 1. Testing jig.

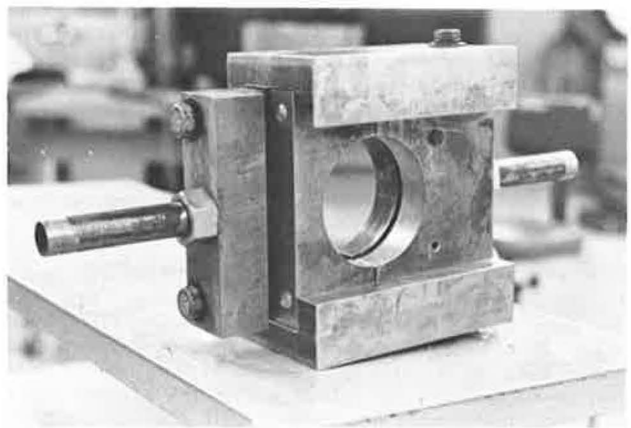

Figure 2. Hydraulic testing machine, jig, and specimen.

Table 1. Summary of bond strengths from various projects.

Project	Avg No. of Cores	Method of Surface Preparation	Bond Strength (kPa)
Black Hawk County (1976 project)			
Nov. 1976	10	Scarify and sandblast	7407
March 1977	10	Scarify and sandblast	5786
March 1979	7	Scarify and sandblast	4931
Clayton County (1977 project)			
1977	13	Scarify	2897
1977	14	Water blast	3855
1977	16	Sandblast	4083
1980	6	Sandblast	5186
1980	6	Water blast	4317
1980	6	Scarify	2828
1980	3	Scarify and sandblast	4600
Woodbury County (1978 project)			
1978	5	Scarify and sandblast	3979
Pottawattamie County (I-80, 1979 project)			
1979	17	Scarify and sandblast	3793

Note: 1 kPa = 0.145 lbf/in^2.

taken so that the grout is not so thin that it forms puddles in low spots. The grout is brushed on the surface by using a long-bristled broom. This brushing action aids in assuring a uniform coverage of the surface as well as in preventing any areas of puddling or excessive grout.

On the last concrete-resurfacing project on I-80, the contractor developed a system of spraying the grout on the surface rather than of brushing it on. In addition, he elected to use a mixture of only cement and water. This mixture had been previously evaluated in the laboratory and found to provide the same bonding characteristics as the cement-sand grout.

The cement-water grout was proportioned at the rate of 26.5 L (7 gal)/bag of cement, which converts to a water/cement ratio of 0.62 by weight. This grout was proportioned and mixed in one of the drums at the central mix-proportioning plant and was transported to the job site in an agitator-type truck. The truck was modified by placing pieces of rubber belting on the paddles so as to ensure continuous wiping of the bottom and sides of the truck box, thus preventing any settling of the cement from the mixture.

After various trials, a nozzle was selected that provided an even distribution of grout across a spray width of approximately 1-1.2 m (3-4 ft). This procedure appeared to work very satisfactorily and was considerably less labor-intensive than the method used on previous resurfacing projects in which laborers brushed the grout onto the existing surface.

The timing of grout placement is considered to be critical in achieving the maximum bond. It is most important that the grout be placed immediately ahead of the concrete and that the fresh grout be covered with concrete as soon as possible, before it has been allowed to dry.

SUMMARY AND CONCLUSIONS

After four projects, successfully completed over a four-year period, the techniques and methods needed to construct a bonded PCC resurfacing have been tested. The required procedures, although somewhat unique, are not particularly difficult. Any reasonably competent concrete paving contractor would be able to construct a bonded concrete-resurfacing project.

In review and evaluation of the projects constructed to date, bonded PCC resurfacing is considered a viable alternative to bituminous resurfacing for concrete pavement rehabilitation and restoration.

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REFERENCES

1. L.T. Norling. Concrete Overlays and Resurfacing--A Status Report. Portland Cement Association, Chicago, IL, Jan. 1976.
2. A.A. Anderson. Experimental Project, Thin Bonded Concrete Resurfacing, U.S. 34, West Burlington, Iowa. Portland Cement Association, Chicago, IL, 1956.
3. B.C. Brown and J.V. Bergren. Evaluation of Concrete Bridge Deck Resurfacing in Iowa. Iowa Department of Transportation, Ames, April 1975.
4. C.J. Schroeder, R.A. Britson, and J.V. Bergren. Bonded, Thin-Lift Non-Reinforced Portland Cement Concrete Resurfacing. Iowa Department of Transportation, Ames, May 1977.
5. M.L. Johnson. Bonded Thin-Lift, Non-Reinforced Portland Cement Concrete Resurfacing. Iowa Department of Transportation, Ames, Project HR 191, 1978.
6. Thickness Design for Concrete Pavements. Portland Cement Association, Chicago, IL, 1978.
7. R.W. Gillette. A 10-Year Report on Performance of Bonded Concrete Resurfacings. HRB, Highway Research Record 94, 1965, pp. 61-69.

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