G Present and Future Needs, Priority Programming

G.1 Pavement Performance Prediction and Modeling

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G.2 Alternative Network-Level Algorithms

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G.3 Forecasting Pavement Rehabilitation Needs

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CONCEPTS OF PAVEMENT PERFORMANCE PREDICTION AND MODELING

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CONCEPTS OF PAVEMENT PERFORMANCE PREDICTION AND MODELING

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Modeling of pavement performance is absolutely essential to pavement management on all levels: project level to national network level. There are several types of performance and, correspondingly, several types of performance model. Performance, in its broadest sense, is predicted by deterministic and probabilstic models. The deterministic models include those for predicting primary response, structural, functional, and damage performance of pavements. The probabilistic models include survivor curves, Markov and semi-Markov transition processes. Damage models are particularly important because they impact load equivalence factors, cost allocation, and a variety of other tax related subjects. The principles underlying each of these models, the selection of their mathematical form, the role of the principles of statistics and mechanics in developing an efficient model, the data needed for each model, the modification of each model to represent the effects of maintenance, the limitations and the uses of each model are discussed in this paper.

Many highway agencies have developed one or more of these types of models for different uses in managing pavements. Some of these are very simple and limited in their applications. Other models are comprehensive and well-suited for a broad range of applications. Project level models are different from and more detailed than network level models for they are used in the analysis and design of pavements, of life-cycle cost analyses of alternative designs, and other related purposes. Network level models are necessarily less detailed but are used in the selection of optimal maintenance and rehabilitation strategies, size and weight and cost allocation studies, and network level trade-off analyses between pavement damage, maintenance, and user costs.

Well developed performance models, resting on the twin pillars of statistics (experimental design) and mechanics will satisfy both the technical and economic requirements for managing pavements. The development of better performance models should be a continuing task and much remains to be done.

INTRODUCTION

Monitoring pavement performance, in its broadest sense, has one major purpose: to determine objectively the current condition of pavements and their historical trends so as to use that information in formulating a management plan of action. The term management is used here in its broadest sense to include all of the planning and decision making that is related to the maintenance, rehabilitation, construction, and reconstruction of pavements. Monitoring and inventory activities provide the data; modeling provides the tools for analysis, design, planning, projecting, trade off analysis, ranking and optimizing the choice of alternatives, allocating costs and apportioning funds.

Modeling the performance of pavements is an absolutely essential activity of pavement management, and many highway agencies have developed a variety of pavement performance models for use in their pavement management activities, sometimes paying attention to one type of performance or one type of model to the exclusion of others. However, all types of performance are important and all types of models are useful in predicting at least one kind of performance. This paper gives a brief but comprehensive review of the types of performance, the concepts underlying pavement performance prediction models, the data required as input to them, their uses and their limitations.

DEFINITION OF PERFORMANCE

Performance is a general term for how pavements change their condition or serve their intended function with accumulating use. Performance means something different when it is used at project level or on a network level. And performance changes its meaning again if the network is at the level of a district, a state or province, or of a nation.

At project level, performance is defined by the distress, loss of serviceability index and skid resistance, loss of overall condition, and by the damage that is done by the expected traffic.

At district network level, performance is defined not only by the condition and trends of individual projects but also with the overall condition of the network and with the level of performance that is provided by each type and functional class of road.

At state or province level, there is less concern with the conditions and trends of individual projects but with measures of the overall condition of the pavement networks in each geographical subdivision, especially as they reflect the needs for present and future funding, the effects on user costs, and overweight fees.

The performance of the national network is almost solely concerned with matters of policy and economics, especially in regards to cost allocation, fund apportionment, and equity in taxation, all of which are affected by the needs of individual States or Provinces.

Each of these networks requires a variety of different kinds of performance model for its proper management. The different types of model will be described in the following section.

TYPES OF PERFORMANCE MODEL

There are two basic kinds of performance model: deterministic and probabilistic. While the deterministic models predict a single number for the life of a pavement or its level of distress or other measure of its condition, the probabilistic models predict a distribution of such events. Deterministic models include primary response, structural performance, functional performance, and damage models. Probabilistic models, include survivor curves and Markov process models. Each of these will be described briefly in the following.

Primary Response Models

These models predict primary responses of pavement to imposed loads and climatic conditions, such as deflection, stress, strain, thermal stress, water content, both frozen and unfrozen, and temperature. These models may be either mechanistic, empirical, or mechanistic - empirical models which have been calibrated with observed field data.

Structural Performance Models

These models predict pavement distress of all sorts and composite measures of pavement condition such as the pavement condition index. These models may be empirical or mechanistic - empirical. No entirely mechanistic model of distress exists at present but there is no reason why they cannot be formulated and developed.

Functional Performance Models

These models predict the present serviceability index, pavement surface friction, and wet-weather safety index. All of these are measures of the <u>function</u> of pavements to carry the traveling public in comfort and safety.

Damage Models

These models are derived from either the structural or functional performance models, and it is from damage models that load equivalence factors are determined.

"Damage" is normalized distress or loss of serviceability index. Damage starts at zero and becomes 1.0 when an unacceptable level of distress of serviceability is reached. The damage equation used at the AASHO Road Test is of the form

$$g = \frac{p_i - p}{p_i - p_t} = \left(\frac{W}{\rho}\right)^{\beta} \tag{1}$$

where g = the "damage" index after the passage of W standard loads or equivalent standard loads,

p_i = the initial serviceability index,

p_t = the terminal or unacceptable level of serviceability index,

p = the serviceability index after the passage of W standard loads,

W = the number of standard loads or equivalent standard loads,

ρ, β = constants which depend upon the structural design of the pavement, the stiffness of the subgrade, the magnitude of the load, and the climate.

The damage index, g, begins at zero when W equals zero and p is equal to p_i. It becomes 1.0 when W equals ρ and p is equal to the terminal serviceability index.

Other types of damage can be defined in a similar way. For example, both the area and the severity of distress may be made into damage indices by dividing them by the maximum acceptable level of these:

$$g = \frac{a}{a_t} \tag{2}$$

or

$$g = \frac{s}{s_t} \tag{3}$$

where a = percent of the total area of a pavement that is distressed,

a_t = "terminal" or maximum acceptable percent of the total area of distressed pavement,

s = the severity level of the distress,

s_t = "terminal" or maximum acceptable severity of pavement distress

A performance equation (structural or functional) can be converted into a damage equation by dividing it by the range of acceptable values of distress or serviceability index. As stated above, the importance of damage models is the fact that load equivalence factors are derived from them. These, in turn, make it possible to design pavements structurally to withstand the effects of mixed traffic and to assist in estimating the equitable share of the cost of the construction and rehabilitation of pavement that should be borne by each level of load.

Load Equivalence Factors. A load equivalence factor is a ratio of the number of load applications of a standard load to cause a defined level of damage to the number of load applications of another load to cause the same level of damage. In the AASHO Road Test, the standard load was established to be the 18-kip single axle load. The number of load applications of this load, N_{18}, which causes a level of damage, g, is given by

$$N_{18} = \rho_{18}(g)^{1/\beta_{18}} \tag{4}$$

Similarly, the number of load applications of another load, j, to cause the same level of damage is

$$N_j = \rho_j(g)^{1/\beta_{18}} \tag{5}$$

The load equivalence factor for the second load is defined as

$$(L.E.)_j = \frac{N_{18}}{N_j} \tag{6}$$

The load equivalence factors derived from two different forms of equation will be different. Also, for any given load, there will be a load equivalence factor for each type of damage that can be identified on a pavement. Thus, for any given load, there is a load equivalence factor for serviceability index loss such as was developed at the AASHO Road Test, but there is also a load equivalence factor for alligator cracking, rutting, joint faulting, pumping, and so on. None of these is equal to any other and all are dependent on the assumed form of the damage equation.

The importance of load equivalence factors in design and in cost allocation insists that the choice of the form of equation from which load equivalence factors are developed should not be arbitrary, but should be undertaken carefully, by selecting the form to meet all known physical and mathematical boundary conditions. More will be said of this point subsequently. An alternative to this is to develop mechanistic models to the point where the damage to a pavement due to each load level in mixed traffic may be computed and accumulated directly.

Marginal Load Equivalence Factors. An interesting concept for load equivalence factors has been advanced by Markow (1). Recognizing that the rate at which damage develops in a pavement at any given time depends upon the prior condition of the pavement at that time, Markow suggested that load equivalence factors should be calculated based upon a marginal damage concept, making the range from 0 to 1 be between any two pre-determined levels of distress or serviceability index. In this way, the load equivalence factors for the same vehicle will change as the pavement becomes more distressed, and will also depend upon the presence of other types of distress and upon timely maintenance actions. The objection may be made that this will make the calculation of load equivalence factors more difficult and pavement design for mixed traffic more complicated but the reply is that it will also make it more realistic. Another reply is that replacing load equivalence factors with efficient and well-calibrated mechanistic models may be a more viable alternative. That was Markow's conclusion (1).

Survivor Curves

Survivor curves are used for planning maintenance and rehabilitation alternatives on pavement networks. The construction, maintenance, and rehabilitation histories that are recorded by the state agencies are valuable sources from which to develop survivor curves. A survivor curve is a graph of probability versus time. The probability drops off with time (or traffic) from a value of 1.0 down to zero and it expresses the percentage of pavements that remain in service after a number of years (or passes of a standard load) without requiring major maintenance or rehabilitation. A typical survivor curve is shown in Figure 1. The slope of the survivor curve is the probability density of survival and is also illustrated in that figure. The probability density curve for survival may be constructed from historical data by determining the percentage of pavements that must be maintained or rehabilitated each year after its most recent major repair or new construction.

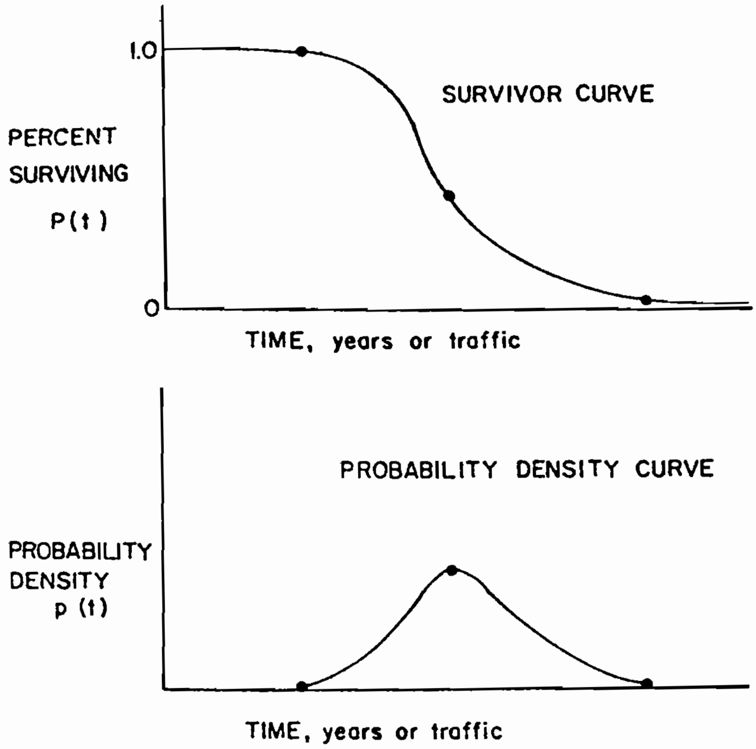

Figure 1. Survivor Curve and Probability Density Function for Survival.

Markov Models of Pavement Deterioration Processes

A Markov transition matrix expresses the probability that a group of pavements of similar age or level of traffic will transition from one state of distress or serviceability index to another within a specified time period. The use of a Markov transition matrix implies that the following assumptions are valid:

1. There are a finite number of states of distress or serviceability index in which the pavement can be found. A "state" is a range of distress or serviceability index such as between a PSI of 4.0 and 4.2.

2. The probability of making a transition from one state to another depends only upon the present state.

3. The transition process is stationary, that is that the probability of changing from one state to another is independent of time. This assumption is a critical one for it assumes that changes in weather conditions within a planning horizon will not affect the transition probabilities. This assumption is not true, in general, for most pavement conditions.

The Markov process describes a probable "before" and "after" condition of the pavement. The "before" condition is described by probabilities that the pavement will be found in each of the assumed finite number of states as is illustrated in Figure 2. The "after" condition is described in a similar manner as illustrated in the same figure. However, the probabilities are shifted downward to lower condition states which are described by ranges of serviceability index.

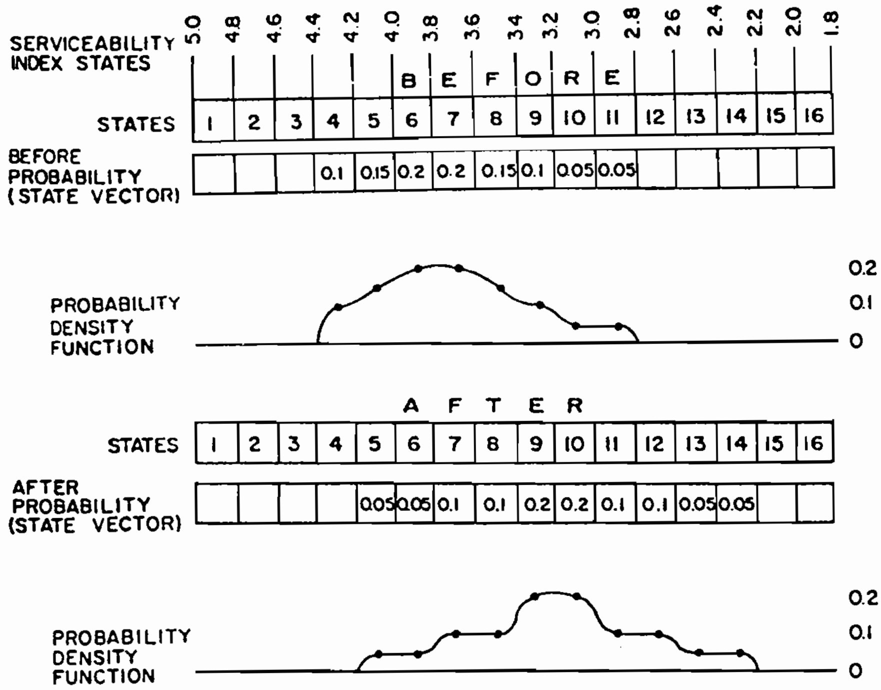

Figure 2. Before and After Serviceability Index State Vectors.

Markov transition matrices can be constructed for any process of pavement deterioration and, especially if the assumptions that are made for Markov processes are valid, they can be used reliably to simulate the overall performance of a network of pavements of similar types with similar weather and traffic patterns.

Semi-Markov Models of Pavement Deterioration Processes

The Semi-Markov processes are identical in every respect with Markov processes except that it is assumed that the process is only stationary during piecewise increments of time. This is actually more realistic since it recognizes that the condition of the pavement and changing weather and traffic conditions cause an alteration in the transition process.

Relations Between Models and Management Levels

As noted previously, some models are used only on particular levels of management. The following table, Table 1, shows which of the types of models just described is used at each level of management. Typically, the higher levels of management are more interested in the probabilistic models and in those which predict composite indexes of the pavement condition.

Table 1. Pavement Management Levels where Performance Models are Used.

Levels of Pavement Management	Types of Performance Model						
	Deterministic Models				Probabilistic Models		
	Primary Response	Structural	Functional	Damage	Survivor Curves	Transition Process Models	
	Deflection Stress, Strain Temperature Thermal Stress Moisture Energy Frozen and Unfrozen Water Content	Distress Pavement Condition Index	Serviceability Index Skid Loss Wet Weather Safety Index	Load Equivalence Marginal Load Equivalence		Markov	Semi-Markov
National Network				✓	✓	✓	✓
State/Provincial Network		✓	✓	✓	✓	✓	✓
District Network		✓	✓	✓	✓	✓	✓
Project	✓	✓	✓	✓			

EFFECTS OF MAINTENANCE ON PERFORMANCE

It has long been the desire of pavement managers and those who model pavement performance to incorporate directly into the prediction models the effect of maintenance when it is applied. Obviously, what is done, the level of effort that is applied, the quality of the work, the extent of the work, the condition of the pavement at the time maintenance is applied, and the rate of change of condition at that time are all very relevant to the result of the application of maintenance. While all of the above are described in qualitative terms, it will be impossible to incorporate them in a model until all of them can be described on a numerical scale. The condition of a pavement and its current rate of change can be calculated using the performance models described above. A numerical scale is needed for maintenance. Such a scale was suggested by Lytton (2). Without presenting all of the detail, the suggestion was to describe maintenance with three numerical "coordinates": level of effort, area, and quality. Level of effort is defined by what is done and where it stands in a table which has a numerical scale of level of effort and positions different maintenance tasks on that scale. Generally, the scale is given by the levels of effort designated in the table below.

Table 2. Maintenance Level of Effort.

Numerical Scale	Application	Maintenance Level
	Routine	
0	Spot	None
1	Spot	Spot Patch & Seal
2	Spot	Preventive
	Programmed	
3	Area	Preservative
4	Area	Corrective
5	Area	Restorative

MODIFICATION OF PERFORMANCE MODELS BY MAINTENANCE

The major modification of deterministic models by maintenance will be a change of the rate of deterioration and simultaneously a possible change in the pavement condition itself. For probabilistic models, such as survivor curves it means only a change of slope. For the Markov process models, it means a change of the probability distribution vector and of the transition matrix. The size of the changes noted above will depend upon: (1) the three maintenance "coordinates"; (2) the current condition of the pavement; and (3) the current rate of deterioration of the pavement. Establishing these relationships in a successful performance prediction model will require carefully and consistently collected sets of field data. Without such data and such models developed from them, it will be difficult to integrate maintenance activities and costs into an overall pavement management plan.

DATA REQUIREMENTS FOR PERFORMANCE MODELS

It is impossible to give more than a brief mention of the types of data that are needed for each of the performance models that are described above. The Strategic Highway Research Progam Data Collection Guide (3) requires nearly 200 pages to encompass all of the data that are envisioned to be needed. The basic types of data are inventory, monitoring, and costs. Inventory data are those which do not change with time or traffic or represent a prior condition of the pavement. Monitoring data are those which do change with time or traffic and constitute the dependent variables of interest in performance equations. Table 3 lists most of the types of data that are needed to construct each of the types of performance model.

Table 3. Data Required for Each Performance Models.

Data Require for Model	Deterministic			Probabilistic		
				Transition Process		
					Markov	
	Structural	Damage	Functional	Sur-vivor Curves	Stat-ionary	Piece Wise Stat.
Inventory Data						
Geometry, Thick.	✓	✓	✓	✓		
Joint Features	✓	✓	✓			
Drainage	✓	✓	✓	✓		
Age	✓	✓	✓	✓	✓	✓
Prior Condition	✓	✓	✓	✓	✓	✓
Environmental	✓	✓	✓	✓	✓	✓
Prior Traffic	✓	✓	✓	✓	✓	✓
Material Prop.	✓	✓				
Subgrade						
Base, Subbase						
Surface						
Overlays						
Cost*						
Initial Const.						
Rehabilitation						
Maintenance						
Monitoring Data (w/time)						
Distress	✓	✓			✓	✓
Traffic	✓	✓	✓	✓	✓	✓
Deflection	✓	✓	✓		✓	✓
Profile	✓	✓			✓	✓
Surface Friction	✓	✓			✓	✓
Maintenance	✓	✓	✓			
Action	✓	✓	✓			
Costs*					✓	✓
Environmental	✓	✓				
Functional Index		✓	✓		✓	✓

*Recorded but not modeled. Cost models are very important but not in the scope of this paper.

DEVELOPMENT AND USE OF PERFORMANCE MODELS

There are different methods by which performance models are developed: by use of regression analysis, by use of the principles of mechanics, by calibrating mechanistic models to fit observed field data, and in a hybrid empirical process, to develop models of interacting distress in which one type of distress is used as an independent variable in predicting another type of distress. Examples of this latter kind are the use of cracking and pumping to predict joint faulting in jointed concrete pavements. Both deterministic and probabilistic models are developed using empirical and mechanistic - empirical methods whereas there are no purely mechanistic performance models at present. Each of those methods has limitations and different uses which will be summarized below.

LIMITATIONS OF PERFORMANCE MODELS

The principle reason for mentioning limitations of the performance models is the temptation to use them, once they are developed, outside of the range of their intended use, i.e., their inference space. Empirical models must be used very carefully in this regard. Unless the form of the equation has been chosen to satisfy all a priori physical and mathematical boundary conditions, it is unlikely to extrapolate well beyond the range of the data from which they were developed. Mechanistic and mechanistic - empirical models have an advantage over empirical models in this regard. They are able to extrapolate well beyond the data that were used for their calibration and, in fact, generally require less data for their development than do empirical models.

The selection of a form of equation to use for pavement performance models must obey principles that are established prior to the analysis of the data. As anyone who has done regression analysis knows, any assumed form of equation can be used in regression analysis, and the only measures of the adequacy of the equation are statistical measures of how well the assumed model fits the observed data within the range of the data. There are no statistical measures which state which form of equation should have been assumed. It is not sufficient to say that "this equation is better than that one because it has a better coefficient of determination" for that is a statement that is made relative to the observed data, and not relative to the logical structure of the problem. It is not sufficient, or even logical, to say that "this equation is better than that one because it has been in use longer or is better known" for the longevity of its use or its current popularity does not guarantee that it is the appropriate form of equation to use as a performance model. As a basic principle, the form of the equation to be used should be selected based upon whether it adheres to the boundary conditions or other physical principles that govern the growth of pavement distress or roughness or the loss of serviceability index. If there are several equations that meet all of these conditions then, and only then, should the selection among the candidates be made on the basis of some statistical measure of the fit of the data.

The relative size of load equivalence factors has a direct bearing on the taxes paid by each vehicle. If load equivalence factors and thus damage functions and thus the forms of the equation of the damage functions and pavement performance models are so important, it is sufficiently important not to leave the choice of the form of equation up to chance or to an arbitrary choice based on computional convenience or to a selection based purely upon statistical measures of the fit to observed data or to an appeal to the number of years an equation has been used or to its current popularity. Any selection of the form of equation based upon these premises must be rejected a priori and the reasoning behind the selection must be declared "non-persuasive".

The *a priori* conditions that must be met by a pavement performance model are discussed below as they apply to damage functions, which have the same form of equation as the performance models from which they were derived. In this discussion, it is sufficient to state that damage, g, is a function of the number of load applications, N. The slope of the damage functions is dg/dN.

Some of the boundary conditions which should be satisfied by the selected form of damage function are as follows:

1. Initial Value The initial value of all damage is zero.

2. Initial Slope Careful considerations should be given to this. Most damage has a slope, dg/dN, that is initially zero. This is true of cracking and of loss of serviceability index. However, some types damage such as "roughness" or rutting have an initial upsurge, then level off, and finally toward the end of the pavement life, turn upwards again. The second upturn in rutting and other types of distress is commonly covered under the sixth category of boundary conditions, i.e. interacting distress.

3. Overall Trend Most damage is irreversible, i.e. the slope, dg/dN, is always increasing.

4. Variations in Slope Most types of damage that are affected by changes in climatic variables have higher slopes, dg/dN, during the more severely stressing season, usually winter or summer, and this means that the selected damage function should have independent variables which are sensitive to the climate and move the slope, dg/dN, up or down in a manner that is physically realistic.

5. Final Slope In this, the damage functions differ most markedly. Some types of damage, such as loss of serviceability index, or area of distress, or total length of cracking per unit area have a strict upper limit. The total loss of serviceability index can be no greater than the initial serviceability index; the distressed area can be no greater than 100 percent; and if there is a limiting crack spacing, there must be an upper limit on the length of cracking per unit area. In all of these damage functions, the final slope, dg/dN, must be zero. This type of equation approaches a horizontal asymptote. Any equation for which the final slope is not equal to zero as N approaches infinity is, *a priori* unacceptable as a damage function for these types of distress. In other types of damage, such as roughness, there is no such constraint.

6. Interacting Distress The final slope of a damage function may be controlled by an interacting distress type. A clear example of this is when rutting begins to increase markedly toward the end of the pavement life. The physical reason for this is that the pavement has cracked, and in some cases the cracks may not have appeared at the surface of the pavement when the slope of the damage curve, dg/dN, begins to increase. The damage function, to be physically realistic, must include as an independent variable

the progressive cracking of the pavement from that point on and not try to explain it as a change in a material property. Water that enters the pavement through the cracks will soften the layer materials and cause a change in the materials properties that affect rutting. Strictly speaking, when an interacting distress begins to occur in a pavement, it should be treated as a different type of damage that will be affected by loads in a markedly different way than was the distress that preceded it.

7. Final Value The maximum value of damage has an upper limit only for those types of distress for which the final slope, dg/dN, is zero. All other types of damage have no upper limit.

Adhering to these a priori principles in the selection of a damage function is the only way to assure that the load equivalence factors that are calculated with them are rational and physically realistic, and the same can be said of performance models generally.

There are other limitations of the use of performance models all of which are more severe limitations with empirical models than with mechanistic or mechanistic - empirical models. These include bias, multiple collinearity, and the effects of changes with time. "Bias" comes about by having a poorly designed experiment or one in which there was no experimental design at all. A biased model is one in which a variable has a greater or less influence on the model's predictions than it has in reality. Multiple collinearity comes about by treating two variables as independent variables when they are, in fact, highly correlated. A good example of this is time and traffic. A way around this is to use the ratio of the two as an independent variable (e.g. vehicles per day, 18-k ESALS per year). Changes with time are not easily handled by empirical models unless sufficient data have been collected at frequent enough intervals to permit a proper representation of the changes with time (e.g. change of layer moduli with the seasons).

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All of these limitations must be carefully borne in mind when developing a performance model.

USES OF PERFORMANCE MODELS

Performance prediction models are used in numerous ways depending upon whether they are used at project or network level. At the project level, performance models are used to design pavements, to perform life cycle cost analyses, to select optimal designs with least total costs including users costs, and in tradeoff analyses in which the annualized costs of new construction, maintenance, rehabilitation, and user costs are summed for a specific pavement design to determine the best time and pavement condition to perform each.

At the network level, there are numerous uses of pavement performance models including the selection of optimal maintenance and rehabilitation strategies; studies of pavement cost responsibilities for different legal vehicle weights and sizes, tire pressures, and suspension systems; determination of equitable permit fees for overweight vehicles; cost allocation studies using load equivalence factors and, in the future, marginal load equivalence factors; and network level trade off analyses of the optimal level and timing of pavement damage, maintenance, and user costs. All of these network level uses of performance models affect the level of taxation and fees that are required of the traveling public and thus constitute the rational basis for all public investments in highway transportation.

CONCLUSIONS

Performance prediction models are essential to the management of pavements at both project and network level in order to satisfy both the technical and economic requirements for managing pavements. Pavement management is impossible without them.

Performance prediction models can be developed best by adhering to a scientific approach in constructing models. The principles of statistics (experimental design) and mechanics should always be used as the twin pillars on which to rest the form and structure of pavement performance prediction models. The selection of these models should not be made arbitrarily, based only on statistical measures; they serve too important a function for that. Mistakes or arbitrariness in the selection of a model cost money for they can be responsible for the allocation of costs away from those elements of the traffic stream which should bear them in all fairness. In addition to cost allocation questions, poorly constructed models make optimal pavement design and the selection of optimal maintenance and rehabilitation strategies impossible. On the other hand, well done performance models, developed in conformance with a scientific approach as described in this paper, will secure for their users economy, technical efficiency and equity.

Finally, the development of performance prediction models should be a continuing task, aimed at continual improvement and better use of the available data. Mechanistic and mechanistic - empirical models, because of their efficiency in the need for calibration data and their superior ability to extrapolate, are, in the long view, the most cost-effective models to develop. However, in the development of these models, much remains to be done.

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rehabilitation program. In addition, the cost of rehabilitation for each section, the total rehabilitation cost for the network, and the measurable impact of the rehabilitation program on network performance or benefit should also be determined in a network analysis.

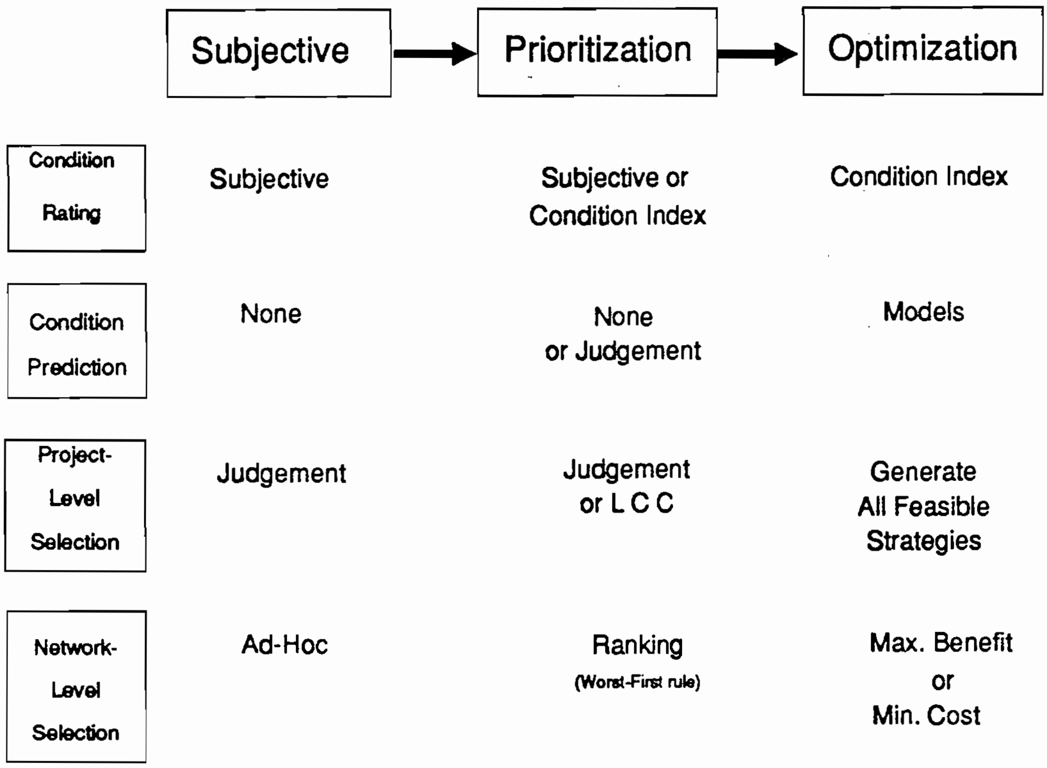

Figure 2.2 - Alternative Network-Level Algorithms.

There are several ways that a rehabilitation program can be generated (see Figure 2.2). The simplest way to arrive at a rehabilitation program involves a subjective inspection of the pavement network (rating each pavement on some scale), identification of pavement sections in need of treatment including a time estimate of when it is needed, and treatment type recommendations. A rehabilitation program is then developed by considering the pavement rating and

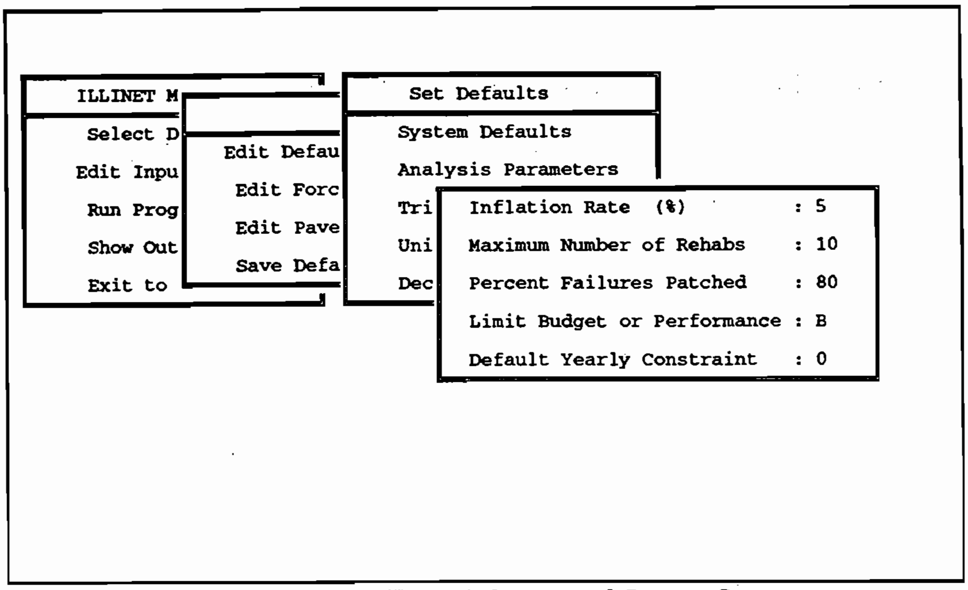

Figure 2.5 - Sample Input Data Menu and Input Screen.

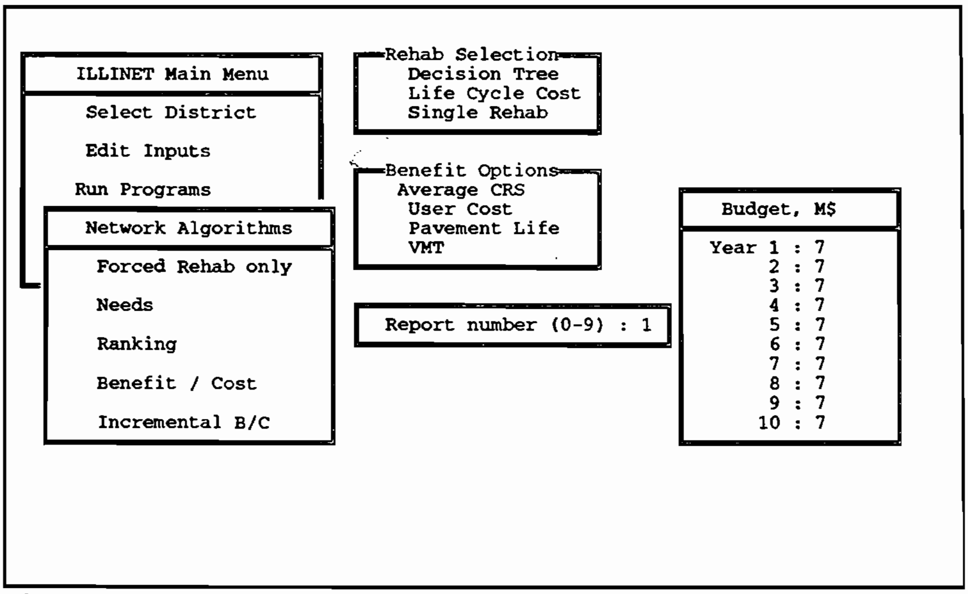

Figure 2.6 - Sample Run Program Menu and Input Screen.

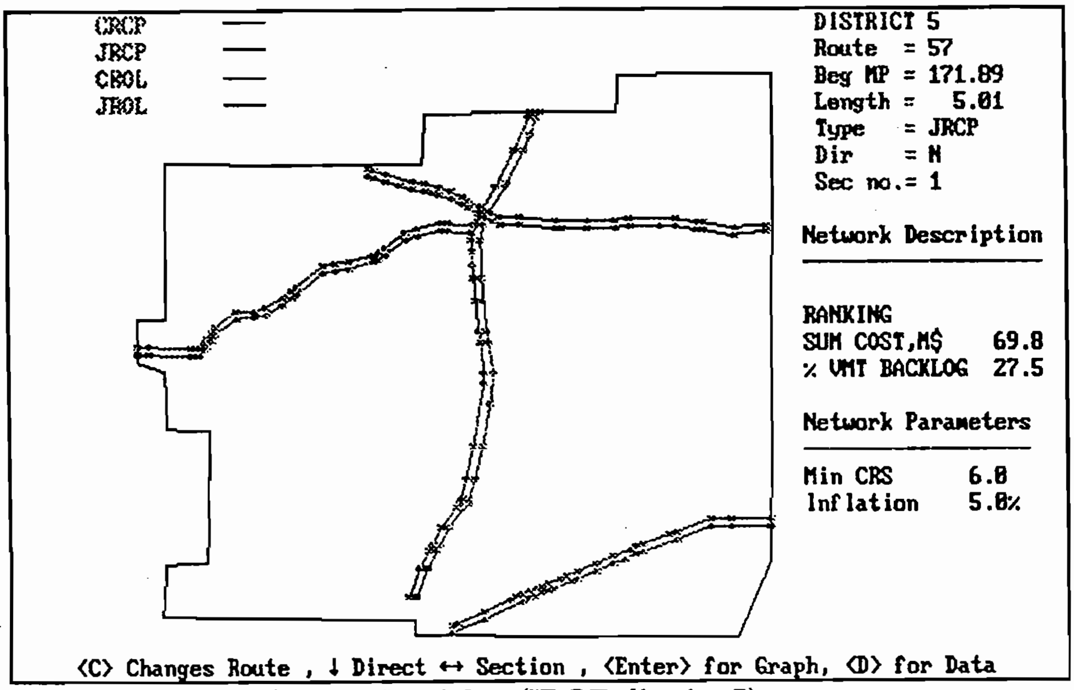

Figure 2.7 - Sample ILLINET Map (IDOT district 5).

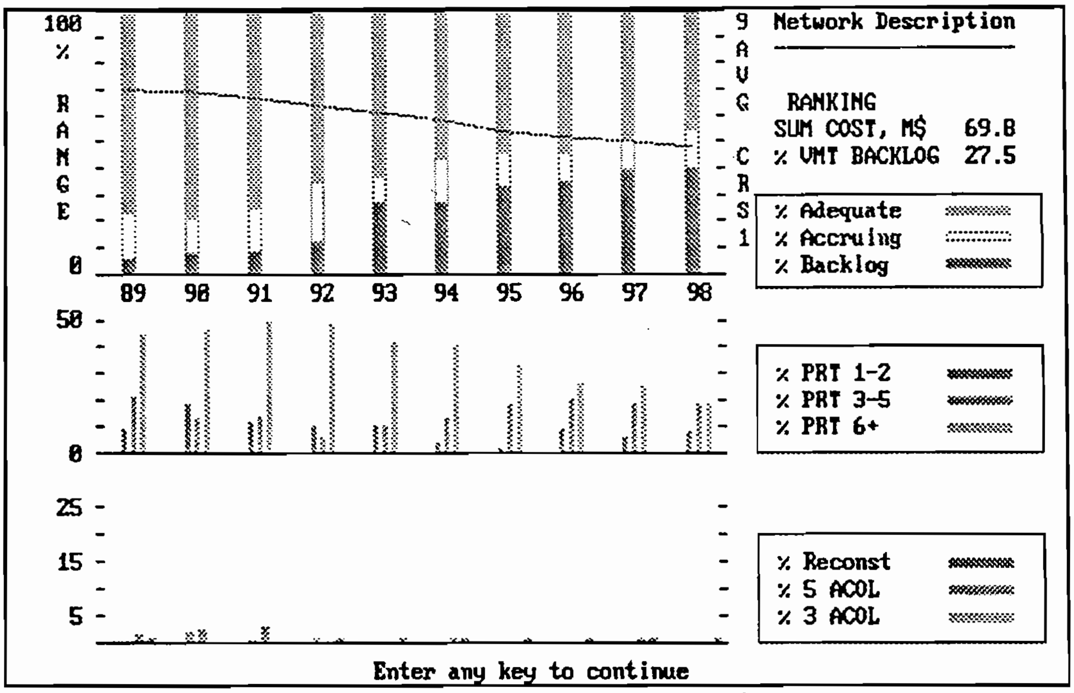

Figure 2.8 - Sample ILLINET Network-Level Graph.

B/C.

The IBC algorithm is accomplished by first calculating an incremental cost (ΔC), an incremental benefit (ΔB), and an IBC ($\Delta B/\Delta C$) for all rehabilitation types (called projects) that apply to a deficient pavement section at a particular year (see Figure 7.5). Only those projects that have a positive IBC are considered, since a negative IBC means that there are no benefits gained by selecting the project over a less costly project. The remaining projects for each section are ranked based on increasing cost and IBC's are graphed (see Figure 7.6). The IBC graph should be concave down as shown in Figure 7.6. When the IBC curve is not concave down the IBC's should be modified to make a concave down curve (see Figure 7.7). The reason projects should have a lower IBC than their previous projects (concave down curve) is that projects are selected incrementally at the network level. Therefore, the benefit and cost of each project should be the sum of incremental benefit and cost of the project itself plus all previous projects for every section. When the IBC of a project (IBC_i) is larger than the IBC of the previous project (IBC_{i-1}) the previous project is set aside and a new IBC is calculated (IBC_n) as follows (Figure 7.7):

$$IBC_n = \frac{(\Delta B_{i-1} + \Delta B_i)}{(\Delta C_{i-1} + \Delta C_i)}$$

If the new IBC is still larger than that of project (IBC_{i-2}) it would be adjusted in a similar manner again.

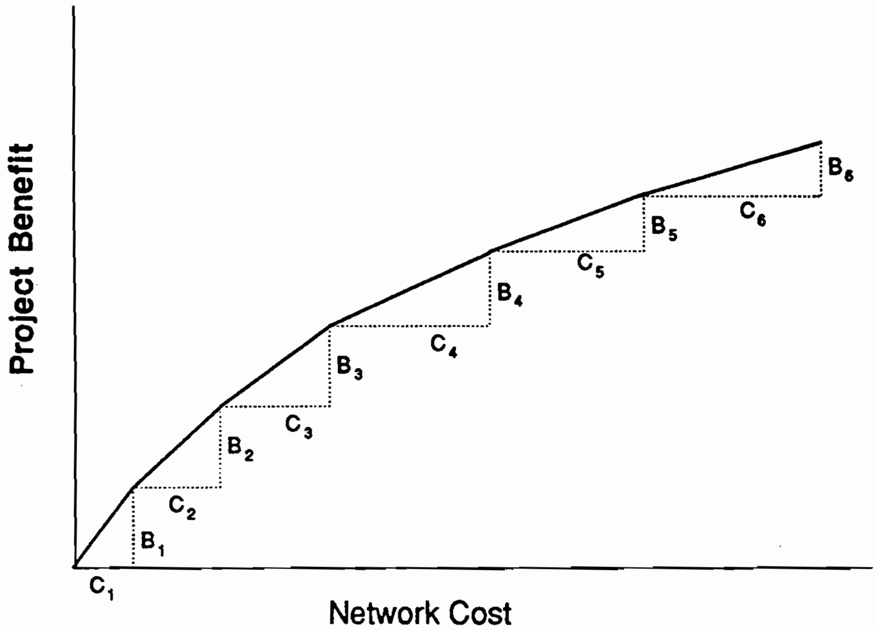

Figure 7.4 - Benefit-Cost Curve for Project Selection for a Given Year.

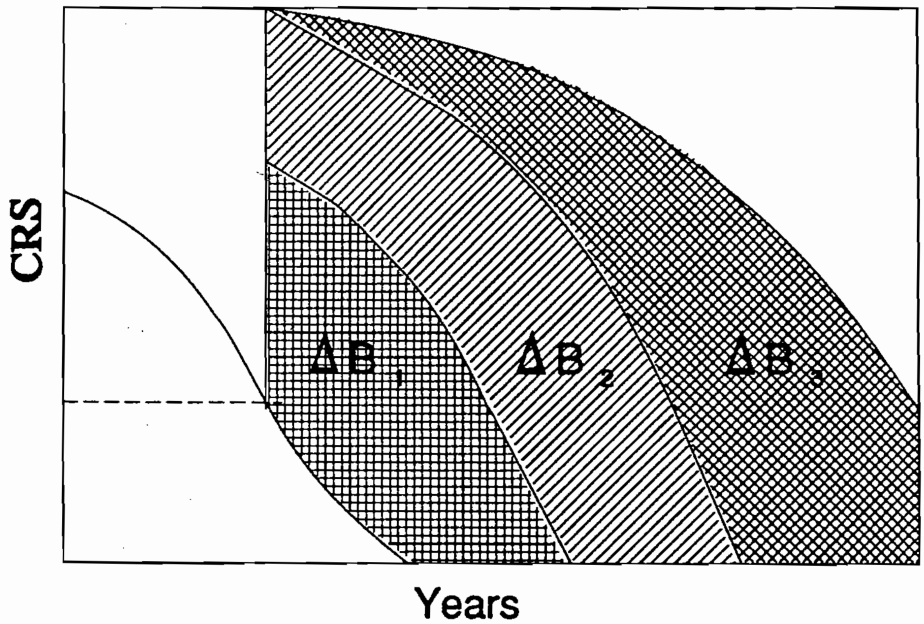

Figure 7.5 - Incremental Benefit and Costs for Different Projects.

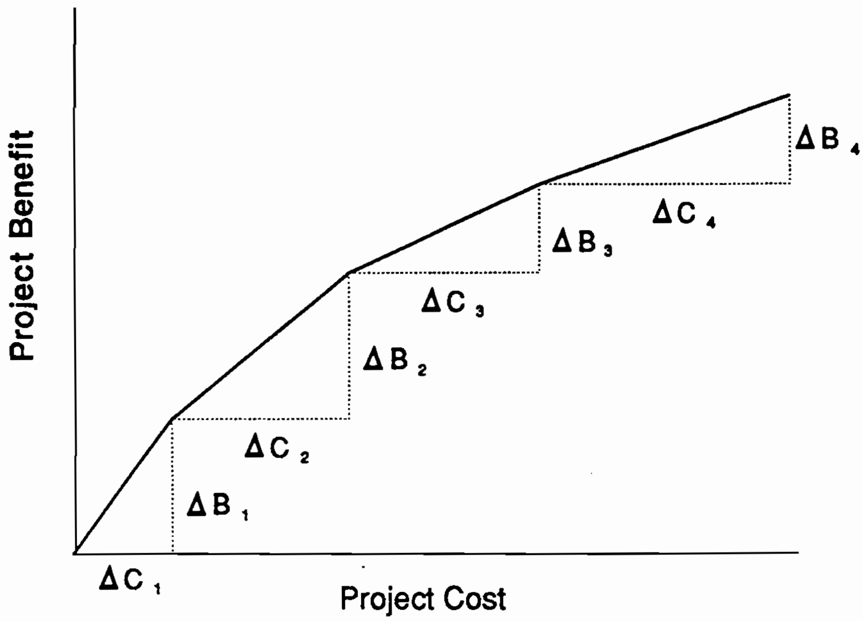

Figure 7.6 - Arranging IBC's for One Section (Concave Situation).

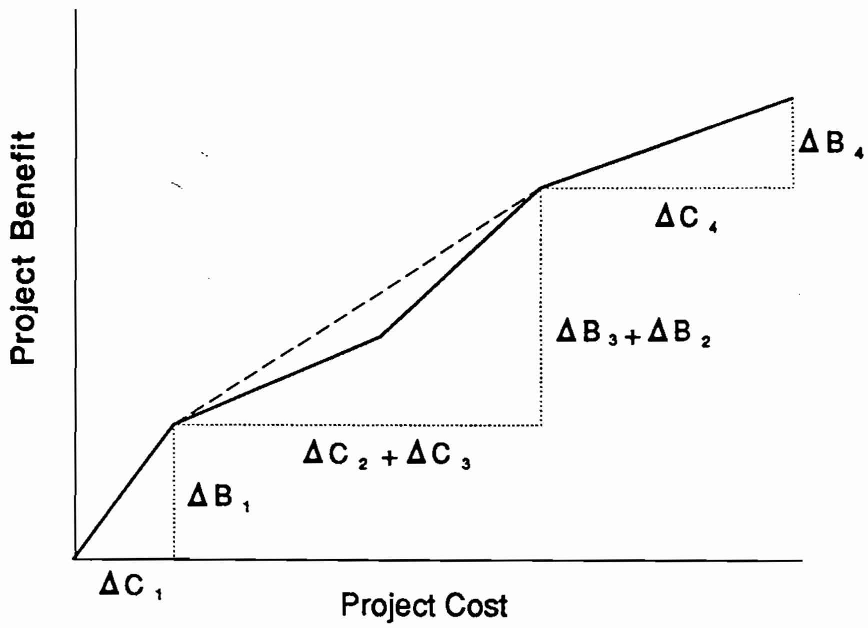

Figure 7.7 - Arranging IBC's for One Section (Non-Concave Situation).

Table 8.2 - Default User Input Values for ILLINET.

Default Parameter	Value	Unit
Analysis Year	1987	
Length of Analysis Period	10	Year
Analysis Interval	1	Year
Trigger for Accruing	6	CRS
Trigger for Backlog	4	CRS
Trigger for Rehabilitation	6	CRS
Inflation	5	Percent
No of Rehabilitations Allowed	1	
Percent Patching	80	Percent
User's Cost for CRS>=6	27	Cents/mi
User's Cost for 6 >CRS> 5	31	Cents/mi
User's Cost for CRS<=5	34	Cents/mi

Table 8.3 - Default Trigger Values for Decision Tree.

	Trigger Values for Rehabilitation			
Rehabilitation Type	BARE JRCP	BARE CRCP	BARE 'D' Cracked	Asphalt Overlays
CPR	6	6	n/a	n/a
3.25-inch Overlay	5	5	6	6
5.0-inch Overlay	4	4	4	4
Reconstruction	3	3	3	3

Table 8.4 - Network Parameters for Six Application Runs for District 5.

Network-Level Opti	RAND	NEEDS	RANK	IBC	OPT	LIN
Project-Level Option	Random	LCC	LCC	All	All	All
Benfit Option	n/a	n/a	n/a	VMT	VMT	VMT
Budget Limit, Million Dollars	75	n/a	75	75	75	75
Cost, Million Dollars	74	90.1	73.8	73.3	75	71.2
Average network CRS 1-9 scale	6.49	7.15	6.74	6.82	6.99	6.81
Average % VMT on Backlog	15.4	2.6	3.5	6.1	4.2	6.2
Remaining Life, Years / mile	3.5	4.7	3.8	4.2	4.4	4.3
% VMT-Backlog @ Year 10	35	10	14	17	14	17
Total CRS Area, CRS-Year / mile	21.5	37.0	26.1	28.4	31.6	29.2
User Benefit, Million Dollars	218	443	287	386	408	383
Total Added Life, Years / mile	2.89	5.9	3.4	4.7	5.2	4.8
VMT on Adequate, Billions	2.98	6.44	3.82	5.64	6.02	5.63
Benefit (VMT-A)/Cost	40	71.5	52	77	80	79

Forecasting Pavement Rehabilitation Needs for Illinois Interstate Highway System

KATHLEEN T. HALL, YING-HAUR LEE, MICHAEL I. DARTER, AND DAVID L. LIPPERT

The Illinois Interstate highway network is deteriorating rapidly because of its age and heavy truck loadings. Unfortunately, the funds required for rehabilitation far exceed the available funds. The Illinois Department of Transportation (IDOT) faces many difficult decisions concerning ranking rehabilitation projects in order of priority and anticipating future pavement conditions and rehabilitation needs. To assist IDOT in making these decisions, three analyses were conducted by using the ILLINET pavement network rehabilitation management program. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the more than 1,200 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's latest multiyear program. The results of these analyses are of immediate practical use to IDOT in forecasting pavement rehabilitation needs for individual pavement sections, Interstate routes, and the entire Interstate network.

The Illinois Interstate highway system consists of about 1,750 two-directional miles of heavily trafficked multiple-lane pavements that were constructed largely between 1957 and 1980. About one-third of these pavements were originally constructed as 10-in. (25.4-cm) jointed reinforced concrete pavement (JRCP), and about two-thirds were originally constructed as continuously reinforced concrete pavement (CRCP) ranging in thickness from 7 to 10 in. (17.8 to 25.4 cm).

These pavements have performed well, despite Illinois' wet-freeze climate, poor subgrade soils, the prevalence of nondurable aggregates, and an unexpectedly high volume of heavy truck loadings. A recent survival analysis indicates that the mean life (years from construction to first major rehabilitation) of these pavements was about equal to the design life of 20 years, whereas the mean 18-kip (8.1-metric-ton) equivalent single axle loadings (ESALs) carried was three to four times higher than the design traffic (*1*).

The Illinois Interstate system is now deteriorating rapidly because of its age and the high volume of heavy truck loadings. As of 1991 about 60 percent of the system had been resurfaced, and much of the rest either is currently in need of rehabilitation or will be within the next 10 years. Unfortunately, the funds required for rehabilitation far exceed the available funds. The Illinois Department of Transportation (IDOT) faces many difficult decisions concerning ranking rehabilitation projects in priority order and anticipating future pavement conditions and rehabilitation needs.

In 1985 IDOT began working together with the University of Illinois to develop the Illinois Pavement Feedback System (IPFS). A major part of the IPFS project has been the development of the IPFS data base, which provides IDOT districts and central offices with data on design, construction, traffic, and condition of 1,263 Interstate highway sections. Although the IPFS data base is neither error-free nor complete, it is sufficiently developed for use in analyses that will provide useful answers to many of IDOT's questions. In addition to the survival analysis already mentioned, other analyses conducted with the IPFS data base include assessment of truck traffic growth rates and the development of performance prediction models.

Another major component of IPFS is the ILLINET pavement rehabilitation network management program. ILLINET uses data from the IPFS data base, decision trees, performance prediction models, and a variety of project-level and network-level management algorithms to generate feasible rehabilitation strategies (treatments and timing) for each pavement section in the Illinois Interstate network for a period of up to 10 years. The network management algorithm options available in ILLINET include analysis of needs (assuming an unconstrained budget), ranking, benefit-cost ratio, incremental benefit-cost ratio, and long-range optimization. The development of ILLINET and its capabilities have been described previously (*2,3*).

Because of the large mileage of Illinois Interstates that will need rehabilitation in the coming years and the expectation that funding for rehabilitation will be inadequate, IDOT is concerned about being able to anticipate the potential impact of insufficient rehabilitation funding on the overall condition of the network. Among the specific questions IDOT would like to answer are the following:

- How accurately can we predict the future condition of individual pavement sections and the future condition of the network as a whole?
- How uniform are the various Interstate routes in condition? Is it feasible to manage long corridors of Interstate as units, or must we continue piecemeal rehabilitation of more than a thousand short highway sections?
- How well are our rehabilitation needs met by the funds available? What will be the effect of the programmed funding level on the overall condition of the network?

Three analyses recently conducted to assist IDOT in answering these questions are described in this paper. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the 1,263 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's latest multiyear rehabilitation program. The purpose of these analyses is to demon-

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strate the practical benefit that a network rehabilitation program with ILLINET's capabilities can provide a state highway agency in quantifying rehabilitation needs and ranking rehabilitation projects in priority order.

ACCURACY OF PAVEMENT CONDITION PREDICTION MODELS

DOT evaluates pavement condition by using condition rating survey (CRS) values, which are assigned by panels of expert raters in field inspections conducted in even-numbered years. CRS is the key pavement condition indicator that is used for planning, programming, and scheduling highway pavement improvement projects. Pavements are rated on a 1 to 9 scale on the basis of the distress observed. The best rating is 9, which is assigned to a newly constructed or resurfaced pavement. For guidance in assigning CRS ratings, panel members consult a manual that illustrates various pavement types and conditions with photographs accompanied by distress descriptions and CRS ratings.

In general, a pavement with a CRS value that falls below 6 would be programmed by IDOT for rehabilitation within the next 5 years. However, many sections have CRS ratings below 6 because their rehabilitation must be deferred because of a lack of funds. Some pavements require considerable maintenance to keep the CRS above 5; below this level ride quality is generally very poor, and maintenance needs become more extensive.

CRS Models

ILLINET contains models to predict CRS for the following pavement types:

- JRCP,
- CRCP, and
- Asphalt concrete (AC) overlay of JRCP (JROL) and CRCP (CROL).

Each predictive model was developed from in-service pavement condition data. After considerable evaluation of different possible model forms, the following functional form was selected for the CRS models:

$$\text{CRS} = 9 - 2 \cdot a \cdot \text{THICK}^{b} \cdot \text{AGE}^{c} \cdot \text{CESAL}^{d} \tag{1}$$

This nonlinear model form may also be expressed in the following linear form by logarithmic transformation:

$$\log_{10}(9 - \text{CRS}) = 0.301 + \log_{10} a + b \cdot \log_{10}\text{THICK} + c \cdot \log_{10}\text{AGE} + d \cdot \log_{10}\text{CESAL} \tag{2}$$

where

CRS = panel condition survey rating (1 to 9),
THICK = slab thickness for JRCP or CRCP and overlay thickness for AC overlay,
AGE = years since construction or overlay,
CESAL = accumulated million ESALs in outer lane since construction or overlay, and
a, *b*, *c*, *d* = constants for each pavement type (Table 1).

TABLE 1 Constants for CRS Model Prediction

MODEL	CRS MODEL CONSTANTS			
	log(a)	b	c	d
JROL, CROL	-0.4185	-0.1458	0.5732	0.1431
JRCP	1.7241	-2.7359	0.3800	0.6212
CRCP	0.7900	-1.3121	0.1849	0.2634

CRS Model Calibration

Within a certain climatic range (i.e., Illinois conditions) pavements of a certain type and design can be expected to exhibit a general trend in condition as a function of time and traffic loadings. However, even pavements of a single type and design can exhibit highly variable performances. Therefore, the prediction model must be calibrated to the observed condition of a specific section to accurately predict the performance of that section.

In other words if the actual current condition of a given section differs from the CRS predicted by the model (as it almost certainly will, because the model describes the mean performance of all sections of that pavement type), then the prediction curve must be adjusted to match the actual value. If this calibration is not done future conditions predicted by the model for that section will not be reasonable.

Two different methods for prediction model calibration are available. The first method basically involves shifting the prediction curve upward or downward so that it passes through and extrapolates from the actual known pavement condition (e.g., CRS). The extrapolated curve is parallel to (and thus predicts the same rate of deterioration as) the mean curve. This approach inherently assumes that the data on age and past accumulated traffic are accurate but that the specific section's performance differs from the predicted mean performance.

The second calibration method uses the actual current condition (e.g., CRS) and the current annual traffic level to "backcast" values for the age or past accumulated traffic inputs, which will predict a condition level matching the actual value. This method, which shifts the mean curve horizontally forward or backward until it passes through the actual known condition level, is particularly appropriate when the accuracy of the age or past traffic data are questionable.

This latter calibration method is currently used in ILLINET because of the uncertainty associated with estimating accumulated ESALs. The current annual ESALs in the outer traffic lane may be estimated more reliably from current or recent counts of the average daily traffic, single-unit trucks, and multiple-unit trucks. A direct relationship is assumed to exist between pavement age, annual ESALs (ESALPYR), and cumulative ESALs:

$$\text{CESAL} = \text{AGE} \cdot \text{ESALPYR} \tag{3}$$

The CRS model for a given pavement type may be calibrated to the current condition of any given section of that type in any year by calculating the following two calibration constants:

$$C_1 = \left(\frac{9 - \text{CRS}}{2 \cdot a \cdot \text{THICK}^{b} \cdot \text{ESALPYR}^{d}} \right)^{\frac{1}{c+d}} \tag{4}$$

$$C_2 = C_1 \cdot \mathrm{ESALPYR} \quad (5)$$

Once the model has been calibrated to the current condition of the section, the condition of the section in any future year may be predicted as a function of the change in the age of the pavement in years (ΔYEAR) and the change in millions of accumulated ESALs (ΔCESAL) over that time period:

$$\mathrm{CRS}_{\mathrm{future}} = 9 - 2 \cdot a \cdot \mathrm{THICK}^b \cdot (C_1 + \Delta\mathrm{YEAR})^c \cdot (C_2 + \Delta\mathrm{CESAL})^d \quad (6)$$

The increase in millions of accumulated ESALs over some future time period is computed by using the current annual ESALs (ESALPYR), the length of time (ΔYEAR), and an assumed annual ESAL growth rate. A compound growth rate of 6 percent is used as a default in ILLINET, although this value may be changed at the user's discretion.

Accuracy of CRS Prediction for Pavements Without D-Cracking

The first step in assessing the accuracy of the CRS prediction models was a comparison of the 1992 CRS values predicted by the models with the actual 1992 CRS values assigned by the expert rating panels. This was done by using CRS history, pavement design, and traffic information retrieved for each of the 1,263 Interstate sections in the IPFS data base.

For each section the appropriate model for the pavement type was calibrated to the actual 1990 CRS, and the CRS was projected from that point assuming a 6 percent compound growth rate in ESALs. This comparison showed that the models predicted CRS well from 1990 to 1992 for bare CRCP, bare JRCP, AC-overlaid CRCP, and AC-overlaid JRCP without D-cracking. The results are shown in Figures 1, 2, 3, and 4, respectively.

To assess how many years into the future the CRS models could provide accurate predictions, the comparison of predicted and actual 1992 CRS values was repeated with models calibrated to 1988 CRS data and then to 1986 CRS data. Sections that were rehabilitated between the starting year and 1992 were excluded from the analysis. The results for pavements without D-cracking indicate that the models' predictive accuracies are good even for 6 years into the future. Analysis of the models' accuracies for longer time periods could be done, but there is a limitation: the predicted and actual CRS values can be compared only for sections that do not receive any rehabilitation during the time period considered. For periods of 8 years or more the number of sections available for use in the analysis becomes considerably smaller.

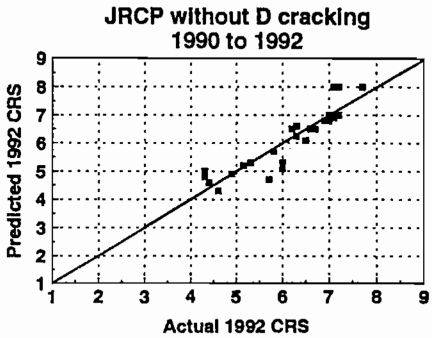

FIGURE 2 Predicted versus actual 1992 CRS for JRCP without D-cracking.

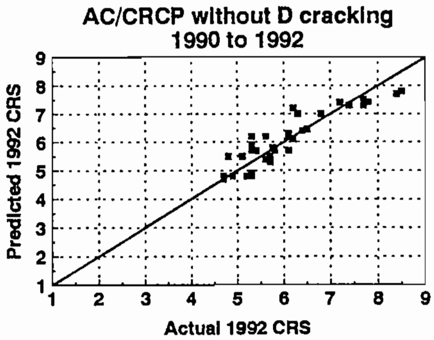

FIGURE 3 Predicted versus actual 1992 CRS for AC-overlaid CRCP without D-cracking.

Accuracy of CRS Prediction for Pavements with D-Cracking

The drop in CRS from 1990 to 1992 was generally greater for D-cracked pavements than the models predicted. When the CRS models were developed in 1986 a D-cracking variable was not in-

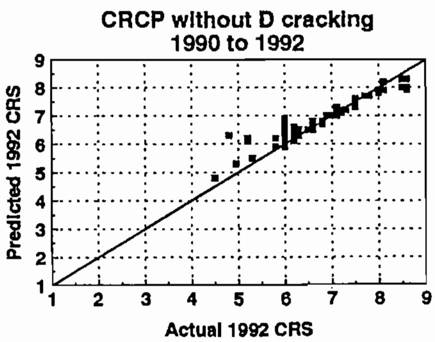

FIGURE 1 Predicted versus actual 1992 CRS for CRCP without D-cracking.

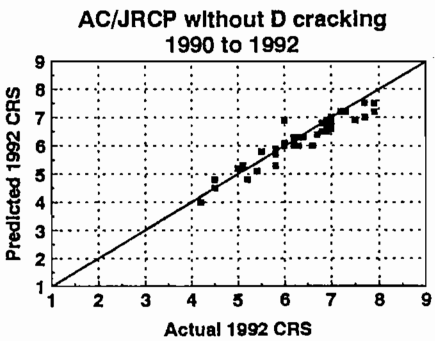

FIGURE 4 Predicted versus actual 1992 CRS for AC-overlaid JRCP without D-cracking.

cluded, primarily because the D-cracking data contained in the IPFS data base at that time were not considered sufficiently reliable.

In 1991 a thorough review of the D-cracking data in the data base was conducted by using distress survey results, materials records, and previous research results. That review was done to conduct survival analyses of bare and resurfaced concrete pavements in Illinois with and without D-cracking (*4*). One finding of the survival analysis was that both bare and overlaid pavements without D-cracking lasted longer and carried more truck traffic than D-cracked pavements of the same type and thickness. The mean life (age and accumulated ESALs) was 20 to 50 percent higher for non-D-cracked pavements than for D-cracked pavements of the same type and thickness.

To account for the more rapid deterioration of D-cracked pavements, an analysis was conducted to determine an appropriate adjustment that could be applied to the predicted rate of loss in CRS. This was done for four pavement categories (bare JRCP, bare CRCP, AC-overlaid JRCP, and AC-overlaid CRCP, all with D cracking) by comparing the predicted with the actual 1992 CRS by using CRS data sets from 1990, 1988, and 1986. The following adjustment factors were found to give the best fit over the time ranges considered:

Adjustment Factor	*Pavement Category*
1.2	Bare JRCP
1.2	AC-overlaid JRCP
1.2	AC-overlaid CRCP
1.5	Bare CRCP

An alternative to applying these adjustment factors to the rate of CRS loss for D-cracked pavements would be to repeat the regression of the CRS models with an additional term for D-cracking. However, the use of adjustment factors may be preferable because IDOT personnel will be able to modify the factors as needed in future years to maintain a good fit of predicted to actual CRS without having to conduct nonlinear regression analyses to modify the CRS models themselves.

REMAINING LIFE ANALYSIS

ILLINET was also used to predict the remaining life of each section of the Illinois Interstate network. The purposes of this analysis were to assess the overall health of the network and to examine the variabilities in the remaining lives of pavements along the various Interstate routes. This knowledge would be useful to IDOT in assessing the feasibility of identifying corridors of multiple sections that could be brought up to uniform condition and subsequently managed as units in terms of future rehabilitation decisions.

Selection of Critical CRS

The "remaining life" of each Interstate section, defined as the number of years from 1993 until the section reached a CRS of 6.0, was predicted by using the CRS models, calibrated to the 1992 CRS and adjusted for D-cracking as described before, and assuming a 6 percent compound ESAL growth rate. This analysis was then repeated by using a CRS of 5.1, which IDOT personnel believed might represent more realistically the level at which a pavement was likely to be rehabilitated (considering the typical budget limitations), even though a CRS of 6.0 was the level at which rehabilitation would be desirable. Of course, the estimate of remaining life depends on the critical CRS selected.

Effect of Maintenance on CRS Prediction

The prediction of the number of years remaining to a CRS of 6.0 is reasonable in most cases; however, the prediction to lower CRS levels for any given section is highly dependent on the level of maintenance applied. Many sections of Interstate highway receive extensive maintenance to keep the pavement in service until rehabilitation can be done. The CRS histories of such sections fluctuate between about 5 and 6 for several years, despite a previous steady decline from 9 to about 6. Of course it is difficult to predict accurately the rate of deterioration for such sections.

Remaining Life of Interstate Routes

The results of the remaining life analysis were plotted by Interstate route and direction. The results for portions of I-55 and I-70 are shown in Figures 5 and 6, respectively, as examples. The heights of the bars indicate the remaining life in years. The numbers on the horizontal axis are mileposts, rounded to the nearest mile, given for reference.

Some Interstate routes show reasonable uniformity in remaining life, whereas others show large variations. I-55 is an example of a route with large variations in remaining life. The nonoverlaid pave-

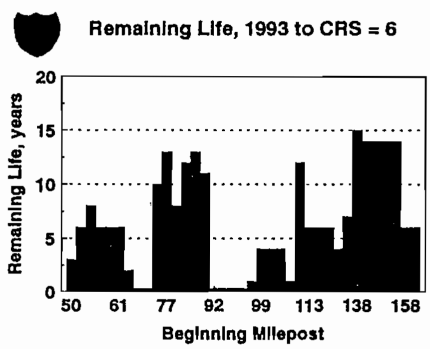

FIGURE 5 Remaining life of pavement sections along portion of Interstate 55.

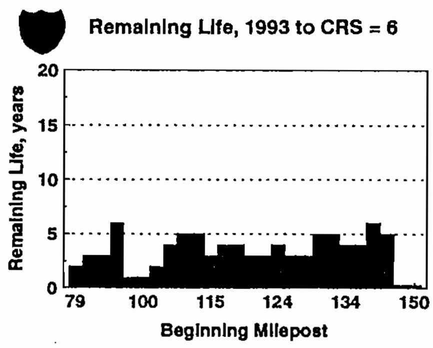

FIGURE 6 Remaining life of pavement sections along portion of Interstate 70.

ment sections represented in Figure 5 range in age from about 15 to 30 years, and the overlays on some sections range in age from about 3 to 12 years. About half of the sections have D-cracking, and thus have shorter predicted remaining lives than sections of similar design and traffic that do not have D-cracking. Some large differences in remaining life by direction are also evident for some sections.

Among the routes with more uniform remaining lives, some have fairly long and others have fairly short remaining lives. I-70 is an example of a route with a uniformly short remaining life: the sections illustrated by Figure 6 are primarily 8-in. (20.3-cm) CRCP with some 10-in. (25.4-cm) JRCP, constructed between 1960 and 1972. Nearly all of these pavements have D-cracking, which, combined with the heavy truck traffic on I-70, has resulted in considerable deterioration of the concrete. All of these sections have been overlaid at least once since 1980, and some have been overlaid three times. It is understandably discouraging to IDOT planners and district engineers to contemplate the future rehabilitation needs of such a long stretch of a heavily trafficked Interstate that, despite frequent rehabilitation and nearly constant maintenance, has only a few more years of remaining life.

Future Analyses of Remaining Life by IDOT

The remaining life analysis capability was added to the ILLINET program so that in future years this analysis can be repeated easily by IDOT personnel for the entire network or specific routes. The user needs only to select an ESAL growth rate and a critical CRS. The standard keyboard "page up" and "page down" keys are used to move through the Interstate route graphs displayed on the computer screen, and once a printer has been selected, the "shift" and "print screen" keys are used to print the displayed graph.

ANALYSIS OF REHABILITATION NEEDS VERSUS IDOT PROGRAMMING

The third analysis conducted was a comparison of the rehabilitation needs predicted by ILLINET and IDOT's proposed multiyear rehabilitation program. This analysis has actually been conducted four times: first with IDOT's improvement program for fiscal years 1991 to 1995 and then for 1992 to 1996, 1993 to 1997, and most recently with the 1994 to 1998 program.

Proposed Highway Improvement Program

The multiyear program itemizes IDOT's proposed expenditures for Interstate highways, state highways, and other facilities in several areas, including pavement rehabilitation, bridge rehabilitation or replacement, major highway construction, and safety improvements. The programmed expenditures considered in this analysis were those for resurfacing and reconstruction of Interstate pavement sections. Programmed expenditures for patching, interchange reconstruction, and bridge reconstruction were excluded.

Rehabilitation Needs Analysis with ILLINET

One of several pavement network management algorithms programmed in ILLINET is the needs algorithm, which estimates the rehabilitation needs for up to 10 years into the future, assuming no yearly budget constraint. Every section in the network whose condition falls below a user-defined minimum CRS is a candidate for rehabilitation. The type of rehabilitation is determined by selection of one of several available options for project-level rehabilitation (*2*). For this analysis the needs algorithm was run by using a single thickness of asphalt resurfacing as the sole rehabilitation strategy. In fact the rehabilitation type is not significant to this analysis, the purpose of which is to predict the timing of rehabilitation, not the cost. The analysis was run for three critical CRS levels: 6.0, 5.5, and 5.1.

Comparison of Rehabilitation Needs with Program by Route

The sections with rehabilitation needs identified by ILLINET and the sections programmed for rehabilitation by IDOT were graphically displayed by Interstate route and direction. A comparison for portions of I-74 and I-80 are shown in Figures 7 and 8, respectively, as examples. For each direction the sections needing rehabilitation according to ILLINET are represented by the bars above the line representing the route, and the sections actually programmed by IDOT for rehabilitation are represented by the bars below the line. The numbers next to the bars indicate beginning and ending mileposts; these are followed in parentheses by the year that rehabilitation is needed or programmed.

A summary of the mileage of rehabilitation needs identified by ILLINET and the programmed rehabilitation mileage is provided in Table 2. This summary indicates that the rehabilitation work pro-

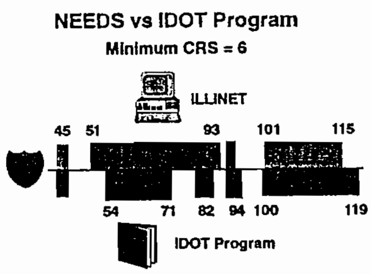

FIGURE 7 Rehabilitation needs (from ILLINET) versus rehabilitation programmed (from IDOT 1994–1998 program) for portion of Interstate 74.

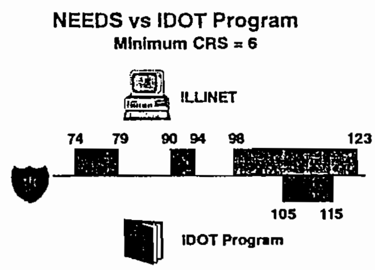

FIGURE 8 Rehabilitation needs (from ILLINET) versus rehabilitation programmed (from IDOT 1994–1998 program) for portion of Interstate 80.

TABLE 2 Summary of Rehabilitation Needs Versus Rehabilitation Program

District	ILLINET Needs			IDOT Programmed Miles	Total Miles Analyzed
	CRS < 6.0	CRS < 5.5	CRS < 5.1		
1	237.17	170.31	153.14	153.81	412.59
2	160.10	114.71	51.55	95.46	320.10
3	196.55	123.85	93.73	117.73	476.63
4	122.83	91.09	72.90	107.27	207.26
5	256.03	146.81	112.90	172.49	510.82
6	106.01	43.57	32.78	62.54	246.56
7	263.95	143.98	113.28	131.63	405.93
8	117.78	86.77	37.62	37.62	352.16
9	110.17	53.82	7.91	7.91	229.88
Total	1570.59	974.91	675.81	939.23	3161.93

Notes:

1. All miles are one-directional.
2. Ratio of miles programmed by miles needed (for critical CRS = 6.0) is 939.23 / 1570.59 = 0.60, or 60 percent.
3. District 2 has one resurfacing project programmed on I-180 (mileposts 5.43 to 9.76, both directions), which was not included in this comparison because I-180 is not currently in the IPFS database.
4. Only resurfacing and reconstruction projects programmed for 1994-1998 were considered in this comparison. Patching, interchange reconstruction, bridge reconstruction, etc. were excluded. Some projects let for bids recently may not be included. The latest bid letting information available was December 1992.

grammed by IDOT with the anticipated available funds is only about 60 percent [939 versus 1,570 mi (1502 versus 2512 km)] of the needs identified by ILLINET to keep all sections of the Interstate above a CRS of 6.

If additional funding is not available a large percentage of Interstate sections are predicted to fall below a CRS of 6.0 over the next 5 years. If the funds available for rehabilitation continue to fall short of the amount required to keep the pavements in acceptable condition the backlog of deficient pavements will continue to grow. This will result in substantial maintenance expenditures and probably more costly rehabilitations as well. Of course what constitutes an acceptable pavement or a deficient pavement depends on the target CRS level selected.

At a critical CRS of 5.5 the ratio is about 96 percent [939 versus 975 mi (1502 versus 1560 km)], and at a critical CRS of 5.1 the programmed mileage exceeds the needs indicated by ILLINET by about 39 percent [(939 versus 676 mi (1502 versus 1082 km)]. These results suggest that the rehabilitation funds programmed over the next 5 years should be sufficient to keep nearly all sections of the Interstate network above a CRS of 5.5 over that time period.

Limitations of Needs Algorithm

ILLINET's needs algorithm was used in the present analysis to identify projects that will reach the selected critical CRS and determine the total mileage of these projects. This algorithm was run by using resurfacing as the single rehabilitation strategy. Hypothetically the budget for rehabilitation is unlimited, so a section is resurfaced as soon as it reaches the critical CRS. This algorithm, particularly when it is run with a single rehabilitation strategy, does not necessarily develop the optimum rehabilitation plan for the network.

Indeed, what is an "optimum" plan depends on what benefit one chooses to maximize or what cost one chooses to minimize. The needs algorithm seeks to eliminate the mileage of deficient pavements. It may do this in a way that is not the most cost-effective for particular sections or for the network as a whole. For example, a severely deteriorated pavement that continues to deteriorate rapidly probably should not be resurfaced every few years; some longer-lasting rehabilitation strategy would be more cost-effective. Other analyses conducted for this research study and described in a separate paper indicate that very different network rehabilitation programs may be developed depending on the network-level management algorithm selected (*5*). For example, in another analysis conducted by using ILLINET, the incremental benefit-cost ratio algorithm produced a network rehabilitation program with the same total cost (in millions of dollars) as the needs algorithm, but with a 50 percent improvement over the needs analysis in vehicle-miles traveled on good pavements. This is because the incremental benefit-cost algorithm may pick more costly rehabilitation strategies for some sections if they are more cost-effective for the network as a whole and also will favor rehabilitation of higher-volume routes, because the benefit that it seeks to maximize is vehicle-miles traveled on good roads.

Future Program-Versus-Needs Analyses by IDOT

The capability of comparing IDOT's multiyear improvement program with the results of the needs analysis was added to the ILLINET program so that in future years this analysis can be repeated easily by IDOT personnel for the entire network or for specific routes. The multiyear program of pavement rehabilitation and reconstruction projects simply needs to be entered into an ASCII input file with route, direction, and beginning and ending milepost data. The user has only to select an ESAL growth rate and a critical CRS.

CONCLUSIONS

The Illinois Interstate highway network is deteriorating rapidly because of its age and heavy truck loadings. Unfortunately, the funds required for rehabilitation far exceed the available funds. IDOT faces many difficult decisions concerning the ranking of rehabilitation projects in priority order and anticipating future pavement conditions and rehabilitation needs.

To assist IDOT in making these decisions three analyses were conducted by using the ILLINET pavement network rehabilitation management program. The first of these was an analysis of the accuracy of ILLINET's pavement condition prediction models. The second was an analysis of the remaining life of each of the more than 1,200 pavement sections in the Illinois Interstate network. The third was a comparison of the rehabilitation needs predicted by ILLINET with those in IDOT's multiyear program.

The analysis of the CRS prediction models showed that future pavement conditions could be predicted with acceptable accuracy for several years into the future. The rate of deterioration for bare and overlaid concrete pavements with D-cracking, which is more rapid than for pavements without D-cracking, could be more accurately predicted by using the adjustment factors determined in the present analysis. However, the effect of maintenance on pavement condition is difficult to predict.

The analysis of the remaining life of the Interstate routes demonstrated considerable variability along some routes and more uniform remaining life along others. This type of information is needed to assess the feasibility of identifying corridors of entire routes or major components of routes that could be brought up to uniform condition and subsequently managed as units in terms of future rehabilitation decisions.

The comparison of rehabilitation needs indicated by the ILLINET software with those in IDOT's multiyear improvement program demonstrated that for any selected critical CRS level a section-by-section and route-by-route comparison of rehabilitation needs and rehabilitation funding could be made. In that analysis the IDOT program met only about 60 percent of the indicated needs when the critical CRS was set at a level below which IDOT personnel generally consider rehabilitation desirable. What constitutes an acceptable or a deficient pavement depends on the critical CRS selected. However, even when rehabilitation costs are deferred because of budget limitations, maintenance costs continue to accrue and increase greatly as the pavement deteriorates.

The purpose of these analyses is to demonstrate the practical benefit that a network rehabilitation program with ILLINET's capabilities can provide a state highway agency in quantifying rehabilitation needs and ranking rehabilitation projects. The graphical displays and graphical printed outputs are useful in communicating the analysis results to the central office and district personnel responsible for rehabilitation planning and programming.

The ILLINET software has also been modified to facilitate these analyses being repeated in the future by IDOT personnel. This represents another step in development of IPFS: after development of the data base, after the retrieval of data for specific analysis demonstrations, and after demonstrating the practical value of the analysis results, user-friendly tools to do those analyses should be put into the hands of the IDOT planners and engineers responsible for pavement rehabilitation decision making. A reliable and accessible data base, reliable performance prediction models, and the tools required to do the analyses needed to support decisions are the essential elements of a dynamic feedback system for continuously improved pavement performance and efficient, cost-effective pavement network management.

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