Evaluation of Performance Based Approach for Determining Contribution of Environmental Factors to Pavement Deterioration H P & R Implementation Study

by

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1.0 INTRODUCTION

1.1 Problem Statement

State Legislation requires that cost responsibility studies be available for the 1991 and 1993 Legislative sessions. Part of this legislative requirement includes the need to re-evaluate and establish a sound basis for the weight-mile tax. To be fair and equitable, this tax should reflect the relative contribution of environmental factors and traffic loading to pavement deterioration. Researchers at Oregon State University recently developed the computer program PBA (**P**erformance **B**ased **A**pproach) for determination of the degree to which pavement deterioration is due to load and non-load related factors (Ordonez and Vinson, 1988). This program is based on the methodology developed by researchers at Purdue University (Fwa and Sinha, 1984) for the State of Indiana's cost allocation study. Evaluation of the PBA methodology and the ability to implement it within Oregon is required prior to attempting to use the method to determine pavement deterioration cost responsibilities.

1.2 Objectives

The objectives of this study were twofold:

1) Determine the validity of the PBA methodology and the viability of using the PBA method in Oregon for evaluation of the proportion of load related and non-load related responsibility in pavement deterioration.

2) Develop a course of action to determine the proportion of load vs. non-load related responsibility in pavement deterioration.

1.3 Project Workplan

Task 1 - Review of Background Research

This task consisted of:

1) review of the performance based methodology research work done by Indiana and that done by researchers at Oregon State University;

2) development of a preliminary opinion of the validity of the PBA method for use by the Oregon Department of Transportation (ODOT); and,

3) recommendation for the work approach for Task 2.

Task 2 - Analysis of Available Data to Determine Validity of PBA Approach

This task consisted of determination of the availability of data for testing the validity of the PBA method.

Task 3 - Recommend Future Direction

Depending upon the outcome of Task 2, this task was to prepare:

1) recommendations for a new cost responsibility analysis approach if insufficient data is available to implement the PBA method or if the PBA method is found to be invalid; or

2) recommendations for an approach to validate the PBA method if the method appears viable but insufficient data is presently available to analyze its validity; or

3) a plan to implement the PBA method if it is determined to be a valid method and there exists sufficient data, or sufficient data could be procured, for its implementation within the legislative time frame.

Task 4 - Final Report

This task consists of preparation of the final report to document all analysis performed and to present recommendations developed in the earlier tasks.

2.0 REVIEW OF BACKGROUND RESEARCH

2.1 Summary of PBA Methodology

The performance based approach to the analysis of pavement damage responsibilities was developed at Purdue University (Fwa and Sinha, 1985) for the Indiana Highway Cost Allocation Study. The limitations faced by Fwa and Sinha when they began development of a method to analyze damage responsibility are much the same as those presently faced by ODOT. Indiana lacked comprehensive pavement damage data in terms of individual types of distress. Time and budget constraints did not allow for development of an disaggregate damage analysis model which is based on damage functions developed for individual types of pavement distress. Indeed, Fwa and Sinha, found in their review of disaggregate damage models, that although such models may be conceptually appealing, there are significant uncertainties as to: 1) the types of distresses to be included in the model; 2) the form of the damage functions associated with each distress; and 3) the relative contribution of each distress to the total pavement damage. These uncertainties impose a significant limitation on the ability to realize the objectivity that the disaggregate analysis approach conceptually offers.

In contrast to Oregon, Indiana did have 5 years of pavement roughness (PCA roadmeter) data covering essentially the entire State highway system. The Indiana Department of Transportation (DOT) also had yearly data on highway maintenance expenditures, in terms of production units, classified by county for each highway. Due to the availability of these data, Fwa and Sinha elected to use an aggregate analysis approach wherein pavement damage is measured by loss in performance.

In the PBA method developed by Fwa and Sinha, theoretical and field performance curves are defined by the plot of Present Serviceability Index (PSI) vs. cumulative traffic loading expressed in 18-kip single axle loads (ESALs), see Figure 1. Loss of performance is defined as the area, measured in PSI-ESAL units (areas A and B in Figure 1), between the initial PSI curve (no-loss line) and a particular performance curve. The AASHTO design equation is assumed to represent the theoretical performance curve based on traffic loading (curve 2 in Figure 1). The PSI values derived from roughness measurements define the field performance curve (curve 1 in Figure 1). By relating historical maintenance expenditures to loss in performance, the effect of maintenance on performance may be determined. This allows determination of a so-called "zero-maintenance" performance curve (curve 4 in Figure 1).

A schematic breakdown of performance loss due to the combined effect of load and environment is shown in Figure 2. The total performance loss area between the no-loss line and the zero-maintenance curve is proportioned among four factors as represented by the proportions, "a", "b", "c", and "d" shown in Figure 2. Performance loss due solely to load effects is represented by the proportion "a" and that due solely to non-load effects is represented by proportion "d". The performance loss due to the interaction of load and non-load effects is represented by proportions "b" and "c", the former representing the load share in the interaction effect and the latter the non-load share. Fwa and Sinha proposed a simple, linear proportioning scheme to separate the load and environmental share of the performance loss due to interaction effects. They proposed that (with reference to Figure 2):

$$b/(b+c+d) = a/(a+b+c+d)$$

$$c/(a+b+c) = d/(a+b+c+d).$$

This assumed proportioning scheme allows for solution of $(a+b)$, the load related proportion of damage responsibility, and $(c+d)$, the non-load related proportion.

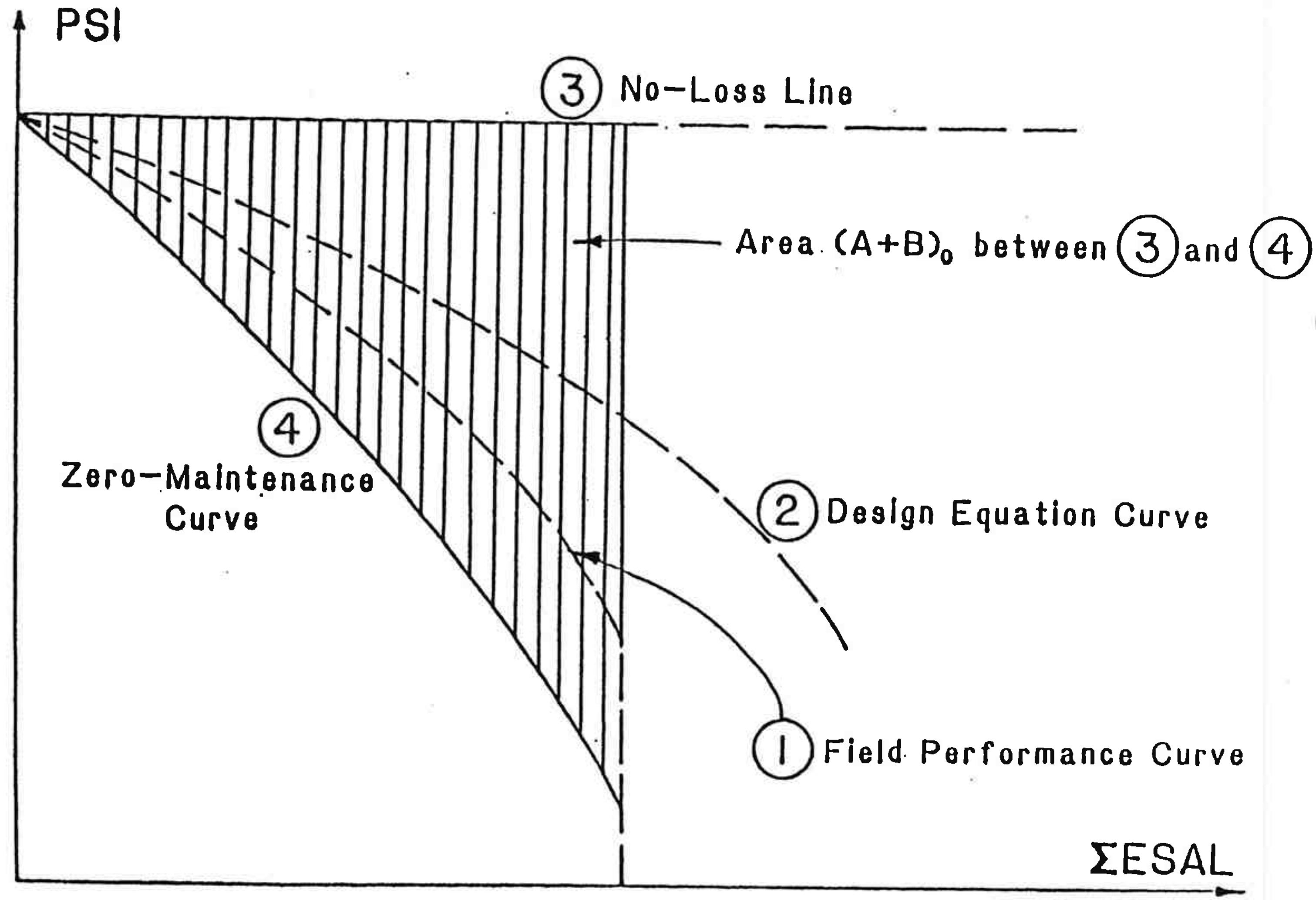


Figure 1 Total Pavement Damage as Defined by Zero-Maintenance Pavement Performance Curves

(Copy of Figure 3.6 from Fwa & Sinha, 1985)

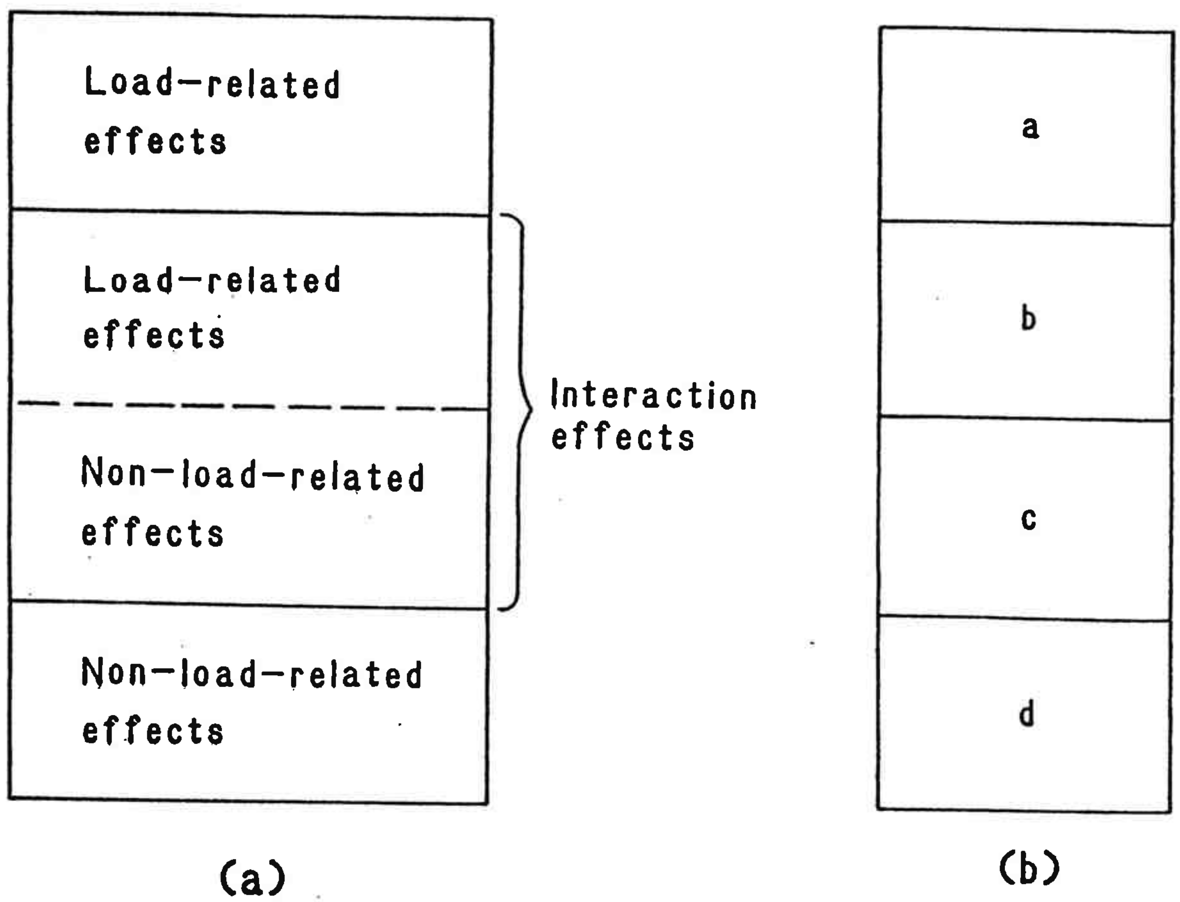


Figure 2 Schematic Diagram Showing Load-Related and Non-Load-Related Effects Responsible for Pavement Damage
(Copy of Figure 3.8 from Fwa and Sinha, 1985)

Since performance loss is quantified in PSI-ESAL units, the allocation of the load related share of pavement damage by vehicle class is logically proportional to the ESAL factor of each vehicle class. Such an allocation is based on the occasioned cost equity concept (Wong and Markow, 1984) in which costs are assigned to users on the basis of the user's contribution to the cost.

2.2 Adaptation of PBA by Oregon State University Researchers

The PBA methodology proposed by the Oregon State University (OSU) researchers has been adapted from the Purdue methodology with only minor changes as noted below. The OSU report (Ordonez and Vinson, 1988) extended the concept of performance loss by replacing the Purdue PSI-ESAL Loss Index with the more general Serviceability-ESAL Index (SEI). This change allows serviceability loss to be measured in terms of PSI, Pavement Condition Rating (PCR) or some measure of structural performance such as pavement deflection.

OSU proposed the use of deflection as an alternative measure of performance since ODOT has a deflection database. To make use of deflection data, they propose using Equation PP.26 from Appendix 2 of the AASHTO Guide to relate pavement deflection for a particular deflection testing device to pavement structural number. Based on the initial properties of the pavement, the theoretical performance curve for deflection as a function of structural number is obtained from Equation PP.26 of the AASHTO Guide. During the life of the pavement, its effective, or "field", structural number may be backcalculated. The field performance curve is then obtained from Equation PP.26 by using the field structural number with the original, or design, values for the remaining variables. Thus, for both the theoretical and field performance curves, deflection is calculated using Equation PP.26 with the only difference being in the value that is used for structural number.

The conceptual relationships between deflection and structural number for the theoretical, field and zero-maintenance level curves are shown in Figure 3 (copy of Figure 12 in Ordonez and Vinson, 1988). The performance losses implied by these relationships are defined in terms of deflection-structural number units rather than SEI units but, otherwise, the method for determining damage responsibilities is the same as that described for damage expressed in SEI units.

If desired, the SEI index can be obtained from the backcalculated field structural number by means of the relationship between the remaining life factor, R_{Lx}, and the pavement condition factor, C_x, as follows:

$$C_x = SN_{exff}/SN_o \quad \text{.............................} \quad 2.2$$

where

SN_{exff} = Backcalculated field structural number

SN_o = Original structural number.

Knowing the pavement condition factor allows the remaining life factor, R_{Lx}, to be determined by either Figure 5.13 of Part III of the AASHTO Guide or by the equation presented in Part CC.5 of Appendix 2. Knowing the remaining life factor allows calculation of the cumulative ESAL loading by the relationship:

$$R_{Lx} = (N_{fx} - x)/N_{fx} \quad \text{.................................} \quad 2.3$$

where

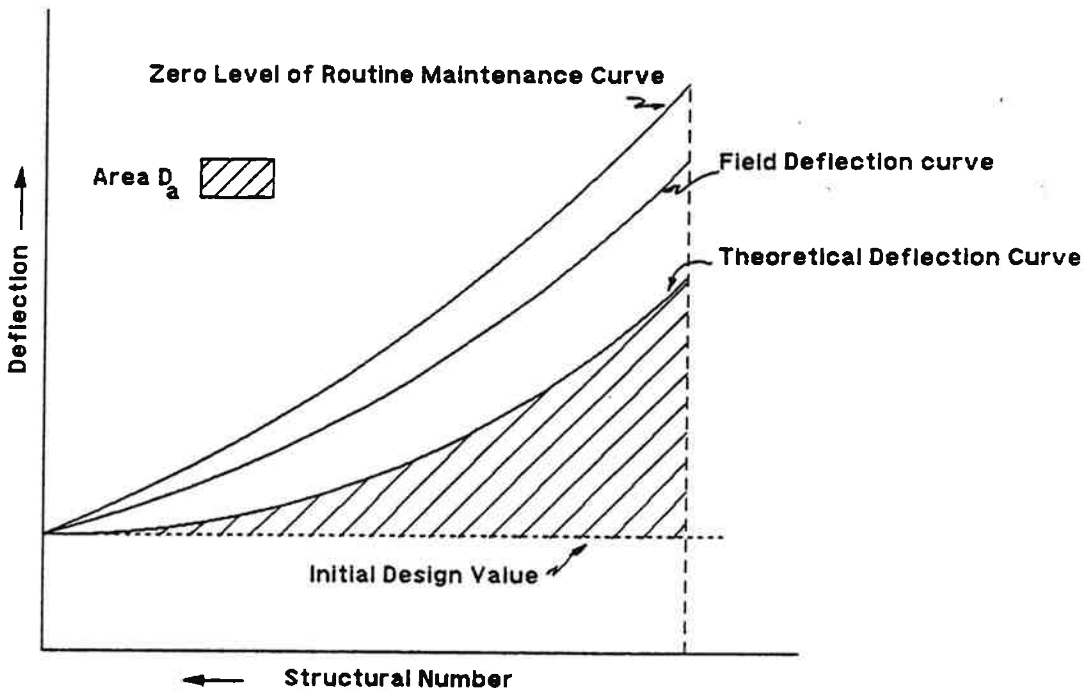


Figure 3: Performance Based Approach Based on Deflection
(Copy of Figure 12 from Ordonez and Vinson, 1988)

N_{fx} = Ultimate traffic capacity of pavement calculated using the AASHTO equation with a terminal serviceability of 1.5 and with a 50% reliability level.

x = Cumulative ESAL loading applied to the pavement

Serviceability loss may also be computed from the remaining life factor using the following equation:

$$P_o - P_x = (P_o - 1.5)*(1-R_{Lx})^{K_L} \qquad \text{2.4}$$

where

P_o = Initial serviceability level

P_x = Present serviceability level corresponding to the traffic loading level x

$$K_L = 0.4 + 1094/(SN_o+1)^{5.19} \qquad \text{2.5}$$

Equation 2.4 is derived directly from the AASHTO flexible pavement performance equation by solving the AASHTO equation for the ratio (x/N_{fx}).

By means of equation 2.4, the field PSI performance curve may be calculated from the backcalculated, i.e., field, structural numbers.

2.3 Key Assumptions of the PBA Methodology

The validity of the PBA method depends upon the following assumptions:

1) The effect of traffic loading on a measure of pavement performance such as Present Serviceability Index, Pavement Condition Rating or deflection can be accurately predicted.

2) The AASHTO design equations satisfy the above assumption and they include only load related effects.

3) The proportionality assumptions made to separate interacting load and environmental effects are valid.

4) A variable can be established to quantify the level of routine pavement maintenance.

Each of these assumptions will be discussed as they relate to Oregon conditions.

Effect of Traffic Loading. Pavement performance models, such as the AASHTO equations, generally relate traffic loading to structural deterioration due to subgrade deformation and/or fatigue cracking originating from the base of the bound layers. Other types of traffic related distresses which are not addressed by these models include:

1) Top down cracking which has been observed to occur in, or near to, the wheel tracks of relatively thick section pavement sections.

2) Studded tire wear.

3) Rutting of asphalt bound layers.

4) Stripping damage in asphalt bound layers.

According to ODOT experience, top down cracking seems to occur most frequently in, or near to, the wheel path. Thus, it appears to be related to ESAL loading but this distress is not predicted by any conventional pavement design or performance models. Studded tire wear is more likely to be a function of ADT rather than ESAL loading. Rutting distress in the asphalt bound layers is a function of tire pressure, ESAL loading and asphalt mix characteristics.

Top down cracking, studded tire wear and rutting of the asphalt bound layers primarily effect the condition of the pavement surface and, hence, will be classified herein as traffic related surface deterioration. Such deterioration is not likely to significantly affect either pavement roughness (longitudinal roughness) or the structural response of the pavement. Hence, traffic related surface deterioration is not likely to be detected, or would be only poorly detected, by roughness measurements or deflection measurements. However, this deterioration would be readily observed in a visual condition survey and, thus, would contribute to the loss in pavement condition rating (PCR). A typical rehabilitation strategy for traffic related surface deterioration is to mill and replace the affected thickness of pavement. Hence, the rehabilitation cost is substantial and, in principal, it should be given the same equity considerations as applied to the allocation of rehabilitation costs for load related structural deterioration. If the traffic related nature of these distresses is ignored, then the rehabilitation costs will be inequitably allocated as though the deterioration were due to non-traffic related factors.

Conceptually, stripping damage is a mix design or quality control problem and, therefore, it is questionable whether it should be included in a cost allocation model based on pavement performance. Current mix design test methods offer promise of better success in detecting stripping susceptibility; however, there are many miles of existing pavement which exhibit this problem. For these pavements, the effect of traffic loading and environment on the rate and degree of stripping damage is probably the most significant factor in their performance. Thus, it can be argued that rehabilitation cost for stripping damaged pavement should be given the same equity considerations as applied to rehabilitation costs for other forms of structural deterioration.

Lottman (1989) has developed the ACMODAS2 & 3 programs which do allow prediction of the effect of stripping damage on pavement performance. Based on laboratory retained tensile strength and modulus test data, these programs will predict rutting and fatigue lives for the asphalt concrete mix. However, due to the detailed data these programs require, they are not suitable for application in the PBA method. Contrary to traffic related surface deterioration, stripping damage will affect the structural condition of the pavement and, in its advanced stages, it may affect the roughness of the pavement. Thus, the effect of stripping damage would be detected, at least to some degree, by deflection or roughness measurements. The consequence of ignoring this form of deterioration is that structural deterioration would appear to progress more rapidly than predicted, hence, some of the deterioration due to stripping damage would be attributed to non-load related effects.

The importance of accurately predicting traffic related surface deterioration and stripping damage in the performance model depends upon the relative magnitude of the costs associated with these types of damage. In contrast to structural deterioration due to subgrade deformation and fatigue cracking, surface deterioration and stripping damage are not general modes of deterioration. They occur on a project specific basis and although they may have a significant cost impact for a particular project, the state-wide rehabilitation cost associated with these forms of distress may be relatively low compared to the rehabilitation cost for structural deterioration.

Whatever the ultimate form of the cost responsibility model, it will likely be, at best, only a first order approximation, eg. the present estimate that 10 percent of pavement deterioration is due to environmental factors was developed on the basis of collective judgement. In this context, it would appear justifiable to simply ignore the traffic related nature of surface condition deterioration and stripping damage. This would result in their rehabilitation costs being allocated under the PBA method as though these costs were due to non-load or non-traffic related effects.

If it is not justifiable to ignore the traffic related nature of surface deterioration and stripping damage, then separate performance models are required for these distresses. Deterioration in structural condition or in serviceability may be related to deterioration in the visual condition of the pavement. As will be discussed later, the AASHTO performance equations can be related to PCR. Rollings and Witczack (1990) developed a structural deterioration model for rigid airfield pavements which relates the Structural Condition Index (SCI), as extracted from the Pavement Condition Index, to structural deterioration. Since surface deterioration is best measured by a visual condition survey and either structural deterioration or serviceability loss may be related to visual condition rating, it appears that visual condition rating (PCR) should be used as the measure of performance for a model that may ultimately incorporate all forms of traffic related deterioration.

Suitability of the AASHTO Equations. The suitability of the AASHTO performance equations for application of the PBA method in Oregon depends upon:

1) whether the AASHTO equations reflect only load induced deterioration; and

2) whether the AASHTO equations accurately predict structural deterioration or serviceability loss for Oregon conditions.

The Road Test was conducted over a period of two years during which minimal maintenance was performed on the test pavement. Accelerated traffic loading was applied during the two year period and most of the pavement serviceability loss actually occurred during the two spring thaws. In effect, the extent of traffic loading that was applied during the most environmentally severe periods is very disproportionate to a normal traffic loading pattern. The test involved a limited number of pavement materials placed over a single subgrade and was located in a single environmental condition.

Despite the above described limitations of the Road Test, it is the best experiment of its kind to date. The design method that evolved from the Road Test is widely used and the data collected at the Road Test have been used to calibrate or validate mechanistic design procedures, eg. the Asphalt Institute's fatigue criteria for asphalt concrete was developed from Road Test data. The AASHTO design equations represent the most widely accepted relationship between traffic loading and measurable performance in terms of serviceability loss. Most other design equations either specify design thickness independent of performance, eg. the Hveem method, or with respect to a terminal structural condition, eg. mechanistic methods which specify the allowable repetitions to limit permanent subgrade deformation to less than a 1/2-inch (Chevron criterion).

The AASHTO equations and the data collected at the Road Test do include environmental effects to some degree. Given the low level of maintenance, the high serviceability loss during periods of extreme environmental exposure and the heavy traffic loading applied in those periods, it is evident that some degree of load and environment interaction effects are implicit in the equations and test data. It should be noted that the above discussed load and environment interaction effects will be implicit in any design procedure or criteria developed from the AASHTO data, eg. mechanistic failure criteria derived from the AASHTO data. However, the duration of the Road Test was too short for environmental effects to have been a significant factor compared to the load effects. Hence, it appears reasonable to conclude that the extent of deterioration due to environmental effects was minimal in the Road Test and may be ignored.

Recent modifications to the AASHTO equations include:

1) recognition of seasonal variation in subgrade modulus by means of the effective subgrade resilient modulus,

2) mechanistic characterization of pavement materials, and

3) recognition of the influence of drainage.

These changes greatly improve the transportability of the AASHTO equations and allow characterization of materials and subgrade to account for local conditions. Essentially, the AASHTO performance equations may be calibrated to local conditions by appropriately characterizing the pavement materials and subgrade.

The question of the accuracy of the AASHTO equations must be viewed with the realization that any pavement performance model is a description of stochastic behavior. Hence, predictions developed from any model are most appropriately stated in terms of the expected result and its associated confidence interval. The stochastic nature of performance prediction is fully recognized in the current AASTHO equations, and confidence limits for performance predictions may be easily constructed. Thus, the theoretical performance curve may be expressed either as the expected performance (at a confidence level of 50%) or as the lower bound of performance for a given confidence level, say 90 %.

The ability to calibrate the AASHTO performance equations to local conditions by appropriate characterization of materials and the ability to statistically define performance are very significant features. As stated earlier, the ultimate form of the cost responsibility model will likely be, at best, only a first order approximation. In this context, it certainly appears reasonable to accept the AASHTO performance equations as the best available models for prediction of the effect of traffic load on pavement performance.

Proportionality Assumptions. In the PBA method proposed by Fwa and Sinha, pavement damage is attributed to four effects:

1) solely load related effects, a main effect;

2) solely non-load related effects, a main effect;

3) the load related effect of the interaction between load and non-load related factors, an interaction effect;

4) the non-load related effect of the interaction between load and non-load related factors, an interaction effect.

To separate the interaction effect into load and non-load related components, Fwa and Sinha assumed that the interaction effects are proportional to the main effects. Although this assumption is logical, it is arbitrary. Examination of the Indiana data for the flexible pavement cases (69 cases) indicates that the average proportions of damage responsibility are allocated as follows:

Indiana Flexible Pavement Data (Fha and Sinha)
Damage Responsibility Proportions, percent

"Pure" Load Related Effects (a)	71.6
Load Related Interaction Effects (b)	19.0
Non-load Related Interaction Effects (c)	4.5
"Pure" Non-load Related Effects (d)	4.9
	100.0

This data indicates that 23.5 percent of the damage responsibility is attributable to interaction effects. Using the same database, Fha and Sinha applied regression analysis to develop a model for predicting the proportion of load related damage responsibility, (a+b). The variables they considered in their model included pavement age, structural number, climate variables and traffic variables. They also considered variables representing the interaction between climate and pavement characteristics, climate and traffic loading, and climate and soil support value. They 0concluded in their analysis that the effect of the interaction variables was not significant and found that the best predictive models were those including pavement structural number and a climate variable. The models they developed, explained between 60 to 75 percent of the variability in the proportion of load related responsibility.

In light of the fact that interaction variables were found to be insignificant in the predictive model, despite 23.5 percent of the damage responsibility being attributed to interaction effects, the proposed damage proportioning scheme does not seem justifiable.

<u>Quantification of Routine Maintenance</u>. Maintenance effort during the life of the pavement will mitigate both load and non-load related deterioration. Therefore, to the extent that maintenance has occurred, the field performance curve will not reflect the total damage. The effect of maintenance is conceptually shown by area between curves 1 and 4 in Figure 1. In order to equitably allocate load and non-load related damage responsibility, the total loss of performance must be determined as represented by the area between no-loss line and zero-maintenance curve 4 in Figure 1.

All other factors being equal, it is assumed that there is a positive correlation between maintenance effort and pavement performance, i.e. the higher the level of maintenance effort, the lower the degree of performance loss. Furthermore, it is assumed that maintenance effort is directly correlated with expenditure.

Fha and Sinha quantified the relationship between maintenance expenditure and performance loss using linear regression. From each route they selected highway sections which had differing levels of maintenance expenditure but otherwise had homogeneous traffic, pavement type, pavement thickness, climate conditions and maintenance technology. Under these circumstances, any differences in performance loss among the sections for a particular route were attributed solely to differences in maintenance expenditure. The performance loss for a particular section was calculated by the PSI-ESAL area between the no-loss line and the field curve. These losses for the various sections along the route were regressed against their respective annual maintenance expenditures per lane mile as shown in Figure 4 (Figure 4.5 from Fha and Sinha). The total performance loss, i.e., the area between the no-loss line and the zero-maintenance curve, for each route is derived from the maintenance expenditure to performance loss relationship as the PSI-ESAL loss which corresponds with zero maintenance expenditure.

Analysis of the maintenance expenditure to PSI-ESAL loss regression relationships indicates that they provide only an approximation to the real relationship and the results do not show a strong linear relationship between PSI-ESAL loss and maintenance expenditure. More than a third of the cases analyzed had four or less data points, i.e. there were four or less sections along a route with differing maintenance expenditure levels. The results suggest that five or more data points per route are desirable.

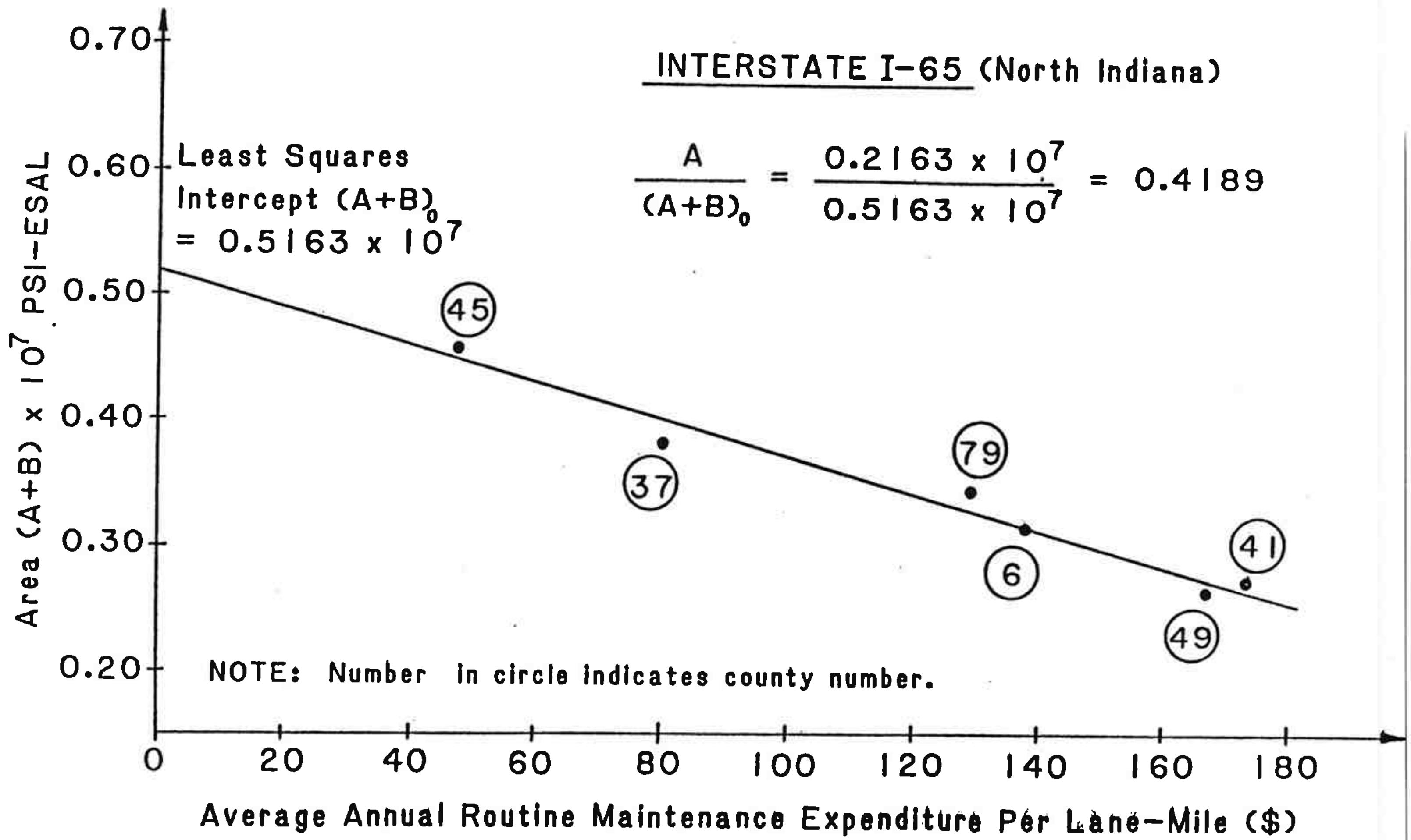


Figure 4 An Example of PSI-ESAL Loss (A+B) versus Routine Maintenance Plot
(Copy of Figure 4.5 from Fwa & Sinha, 1985)

The availability of data for analysis of the relationship between maintenance expenditure and performance loss is severely restricted by the homogeneity requirement that characteristics of all cases included in a particular analysis must be identical except for the level of maintenance expenditure. In Indiana, the length of route within each county is the smallest unit of record keeping for maintenance cost data. Hence, a route must cross several counties and its characteristics must remain homogeneous in order for it to provide suitable data for the maintenance expenditure analysis. In Oregon, the smallest unit of maintenance cost record keeping is the maintenance section foreman's budget. Since maintenance sections usually include several routes, or portions of routes, Oregon maintenance cost data is not directly amenable to the maintenance expenditure analysis as described above.

Fwa and Sinha used the maintenance expenditure-performance loss data to develop a predictive relationship for maintenance effectiveness which is defined by the slope of the regression line shown in Figure 4. Their regression model for this relationship included pavement age, thickness, traffic and environmental variables. They also analyzed the interactions of these variables and found that interaction effects were not significant. Pavement age and soil support variables provided the best predictive model for flexible pavement and, although these variables did not provide the best model for rigid pavement, they resulted in a very reasonable model. These results suggest that the homogeneity requirement may be the result of excluding pavement, traffic or environmental characteristics from the analysis of the relationship between maintenance expenditure and PSI-ESAL loss.

As will be discussed later in the report, it appears possible to overcome the homogeneity restriction imposed by the maintenance cost analysis. This possibility is raised by the results of the predictive models developed from the Indiana data for load-related responsibility and maintenance effectiveness.

2.4 Data Requirements

Essential data required for application of the PBA method consists of:

1) historical pavement performance data

2) historical cumulative ESAL loading

3) pavement inventory data sufficient to allow calculation of predicted performance

4) historical routine maintenance cost data.

Performance loss is measured in terms of the area between performance curves. Therefore, at least 2, and preferably 3, data points should be available to establish the performance curves. Maintenance expenditure records must also be available for the period covered by the performance curves.

In order to analyze the effect of maintenance expenditure on performance and to establish the zero-maintenance performance loss by the Purdue procedure, each route must contain at least two sections having differing maintenance expenditures, but otherwise identical characteristics. Ideally, there should be 5 such sections for each route in the analysis.

The minimum number of routes that should be analyzed for both rigid and flexible pavements depends upon the desired precision in the estimate of the load-related responsibility proportion. The standard deviation of the load-related responsibility proportion as determined from the Indiana data was approximately 6% for both rigid and flexible pavements. If a standard deviation of 10% is assumed and, if it is desired to estimate the load-related responsibility proportion at a 95% confidence interval width of less than 5 percentage points, then a least 18 routes must be analyzed for both flexible and rigid pavement.

The Purdue study concluded that environment is the most significant factor affecting the load responsibility proportion for flexible pavement; whereas, age was found to be the most significant factor for rigid pavement. Mean annual freezing index was found to be the best variable for representing environmental effects for flexible pavements.

In terms of the freezing index, Oregon may be divided into four zones having the approximate ODOT Region and District boundaries as indicated below:

Freezing Index Zones

ODOT Region or District	Approximate Freezing Index Range, degree-days
Regions 1, 2 & 3	0 (West of the Cascades)
Region 4	0 - 250
Districts 12 & 14	250 - 400
District 13	400 - 500

Routes from each of these four zones identified above should be included in the analysis for Oregon. The number of routes selected from each zone should be in proportion to the percentage of the state-wide lane mileage represented within the zone. In Regions 4 and 5, the general gradient of the freezing indices is in an eastwest direction. Hence, routes selected in these regions should be orientated in a north-south alignment to minimize the variation of freezing index along their length.

3.0 ANALYSIS OF AVAILABLE DATA

3.1 Nature of Existing Data

From discussions with ODOT personnel, six sources of historical performance and maintenance data were identified. These sources are:

1. Maintenance Cost Data
2. Ride (Mays Meter) Data
3. Dynaflect Deflection Data
4. Pavement Management System Pavement Condition Ratings
5. Construction Season Reports
6. HPMS Data

There are also twelve pavement sites that are included in the SHRP research program. However, the performance data for these sites is not presently available and may not be available for some time.

Due to schedule constraints, the above sources of data were not comprehensively examined. Rather, the consensus of opinion among ODOT staff was that the maintenance cost data, ride data and pavement condition rating data were the most likely sources to prove useful. Ride data records were matched with maintenance cost records to determine the extent of available data. A listing of the matching ride and maintenance data records that were obtained is presented in Appendix 1.

3.2 Review of Ride and Maintenance Data

Analysis of this data revealed that:

1. Ride data was obtained only in 1984, 1985 and 1989. There is also some concern regarding uniformity of calibration and measurement technique for this data.

2. 51 highway sections were identified as having ride data.

3. Only 34 of the sections have 3 years of ride data records.

4. Yearly maintenance cost data records were found for only 16 of the sections.

5. Of the 34 sections with 3 years of ride data, only 14 of these have maintenance cost data listed for any of the 3 years.

6. 19 of the sections had better ride ratings in 1989 as compared to 1985 without indication that maintenance was performed during the same period.

3.3 Evaluation of Data Constraints

Based on the above findings, it did not appear promising to pursue using historical ride data as a basis of performance to drive the PBA model. It also appears unlikely that maintenance expenditure data for specific highway sections will be available.

Historical maintenance cost data is recorded by the maintenance section foreman's budget. However, the maintenance section usually covers several routes. In order to use this data, it will be necessary to average the maintenance section cost data over the length of highways included in each section. Thus, to compare maintenance expenditure with performance, it is desirable to be able to similarly average the performance data.

Dynaflect deflection data could be used as the basis of performance. However, the length of the deflection test section at a particular site is quite small, typically 1,000 feet. Unless there is a sufficient number of deflection test sites within a maintenance section to allow for averaging of the data, it would be difficult to meaningfully relate the deflection data to maintenance cost records. ODOT pavement design staff have had little success with backcalculation analysis of the Dynaflect data. The ability to perform such analysis is essential if deflection data is to be used as the measure of performance. Hence, it did not appear useful to attempt to use the Dynaflect data.

After elimination of the ride and deflection data, the only remaining historical performance data is the visual condition survey data collected bi-annually for the pavement management system (PMS). One advantage of this data is that ratings of the entire state highway system are available for the period of 1976 to present, i.e., potentially there may be as many as eight ratings available over this period. Having condition ratings for the entire state highway system also should facilitate comparison of rating data to maintenance section cost data.

3.4 Conclusions

Based on the above findings, which are preliminary in nature, it was decided that the visual condition survey ratings appeared to be the best historical measure of performance.

Further examination of maintenance cost records is required to determine the constraints that they impose. However, it appears that only a very limited amount of maintenance cost data for specific highway sections over a period of several ratings is available.

4.0 ALTERNATIVE PERFORMANCE ANALYSIS APPROACH

4.1 Proposed Alternative Approach

As discussed in the previous section, there are several limitations with the historical highway performance and maintenance data available for the Oregon highway system. The performance data is limited to visual condition ratings, although the ratings do cover the entire network and as many as eight ratings may be available over a period of fourteen years. It also appears very unlikely that maintenance expenditure data is available for several homogeneous sections of a given route having differing levels of maintenance expenditure, as required for the PBA maintenance expenditure analysis procedure. Therefore, alternative analysis procedures are necessary in order to apply the PBA methodology using the historical data that is available in Oregon.

An alternative performance analysis procedure and a general predictive model for performance loss are discussed in this section of the report. The use of visual condition ratings in performance analysis is discussed and the basis of the relationship between visual condition and the AASHTO design equations is presented.

Fwa and Sinha generalized the results of their analysis of the Indiana data by development of predictive relationships for the proportion of load-related damage responsibility and by development of predictive relationships for a maintenance effectiveness index. Based on analysis of these relationships, the functional form a general performance loss predictive model is proposed in this report. This relationship would allow the PBA analysis to be performed with maintenance data from several different sections of highway provided that common maintenance technology was used in all sections. Thus, the general performance loss model would obviate the need to have maintenance expenditure data from several identical sections of a highway in order to derive the zero maintenance performance loss for the highway.

4.2 Basis of Visual Condition Performance Methodology

The AASHTO overlay analysis procedure is based on the effective thickness concept and incorporates analysis of remaining pavement life. These two concepts enable visual condition, structural capacity and serviceability to be mathematically related through the AASHTO performance equations. Such relationships are not unique to the AASHTO design procedure. The Asphalt Institute's Manual Series 17 (MS-17) for overlay design includes two approaches to effective thickness design. The first of these procedures graphically relates PSI to pavement effective thickness (Figure V-1 of MS-17). The second procedure, is a component analysis procedure wherein an effective thickness is assigned to the materials within each layer based on visual inspection (Table V-2 of MS-17). This latter procedure appears to be the basis of the visual and structural condition values presented in Table 5.3 of the AASHTO Guide (copy presented herein as Table 1). As earlier discussed, Rollings and Witczak (1990) have recently published their work on development of a structural deterioration model for rigid airfield pavements which relates the Structural Condition Index (SCI), as extracted from the Pavement Condition Index, to structural deterioration.

The fundamental assumption of the effective thickness concept is that the structural properties of the pavement are reduced due to the effects of traffic loading. The structural condition factor, C_x, is the decimal percentage of original structural capacity remaining after a given extent of traffic loading. It is related to the pavement original structural number, SN_o, and the effective structural number, SN_{xeff}, by the equation:

$$C_x = SN_{xeff}/SN_o \qquad \text{.......................................} \quad 4.1$$

In Section 2.2 of this report, the relationship between PSI and structural condition was developed using remaining life as the tool to equate these two measures of pavement performance. Table 1 contains a summary of visual (C_v) and structural (C_x) condition factor values as related to categories of pavement visual condition. According to the special notes presented in this table, visual condition factor values are related to structural condition factor values by:

$$C_v = C_x^{\,2} \qquad \text{.......................................} \quad 4.2$$

Several points should be noted with regard to the values shown in Table 1. Generally, the structural condition factor values agree with thickness conversion factors presented in Table V-2 of MS-17, as might be expected. Hence, the factors are based on the component, effective thickness evaluation procedure. Also, it should be observed that the pavement visual condition factors apply for load related distresses only, i.e., the predominant weight in determining the pavement structural condition factor from the visual condition factor should be given to alligator cracking distress, rutting distress and pavement material deterioration.

Table 1 Copy of Table 5.3

From the 1986 AASHTO Guide for Design of Pavement Structures

Layer Type	Pavement Condition	C_v Visual Condition Factor Range	C_x C_v Struct Cond Factor Value
Asphaltic	1. Asphalt layers that are sound, stable, uncracked and have little to no deformation in the wheel paths	0.9-1.0	.95
	2. Asphalt layers that exhibit some intermittent cracking with slight to moderate wheel path deformation but are still stable.	0.7-0.9	.85
	3. Asphalt layers that exhibit some moderate to high cracking, have ravelling or aggregate degradation and show moderate to high deformations in wheel path	0.5-0.7	.70
	4. Asphalt layers that show very heavy (extensive) cracking, considerable ravelling or degradation and very appreciable wheel path deformations	0.3-0.5	.60
PCC	1. PCC pavement that is uncracked, stable and undersealed, exhibiting no evidence of pumping	0.9-1.0	.95
	2. PCC pavement that is stable and undersealed but shows some initial cracking (with tight, non working cracks) and no evidence of pumping	0.7-0.9	.85
	3. PCC pavement that is appreciably cracked or faulted with signs of progressive crack deterioration: slab fragments may range in size from I to 4 sq.yds., pumping may be present	0.5-0.7	.70
	4. PCC pavement that is very badly cracked or shattered into fragments 2-3 ft. in maximum size	0.3-0.5	.60
Pozzolanic Base/ Subbase	1. Chemically stabilized bases (CTB, LCF...) that are relatively crack free, stable and show no evidence of pumping	0.9-1.0	.95
	2. Chemically stabilized bases (CTB, LCF...) that have developed very strong pattern or fatigue cracking, with wide and working cracks that are progressive in nature: evidence of pumping or other causes of instability may be present	0.3-0.5	.60
Granular Base/ Subbase	1. Unbound granular layers showing no evidence of shear or densification distress, reasonably identical physical properties as when constructed and existing at the same "normal" moisture - density conditions as when constructed	0.9-1.0	.95
	2. Visible evidence of significant distress within layers (shear or densification), aggregate properties have changed significantly due to abrasion, intrusion of fines from subgrade or pumping, and/or significant change in in situ moisture caused by surface infiltration or other sources	0.3-0.5	.60

Special Notes:

I. The visual condition factor, C_v is related to the structural condition factor, C_x by:

$$C_v = C_x^2$$

2. The structural condition factor, C_x and *not* the C_v value, is the variable used in the structural overlay design equation (for all overlay-existing pavement types). It is defined by:

$$SC_{xeff} = C_x SC_o$$

The relationship between the structural condition factor, C_x, and the remaining life factor, R_{Lx}, is discussed in Appendix CC of the AASHTO Guide. The theoretical relationship between remaining life, R_{Lx}, and condition factor, C_x, is given by (Appendix CC.3):

$$C_x = (R_{Lx})^{0.165} \qquad \text{.................................... } 4.3$$

The theoretical remaining life relationship rather than the modified remaining life relationship (Appendix CC.5) appears to be better for the present purpose, since the modified relationship is based on an arbitrary assumption as to the performance of the pavement below a condition factor of $C_x \leq 0.75$ and was designed to give a minimum condition factor of 0.3.

Combining equations 4.1 and 4.3 results in the relationship:

$$C_v = (R_{Lx})^{0.33} \qquad \text{.................................... } 4.4$$

As earlier noted in equation 2.3, remaining life is related to the cumulative traffic loading (ESALs), x, and the traffic loading capacity (ESALs), N_{fx}, of the pavement section by:

$$R_{Lx} = (N_{fx} - x)/N_{fx}$$

$$= (1 - x/N_{fx}) \qquad \text{.................................. } 4.5$$

Hence, the pavement visual condition factor is related to the cumulative traffic loading and capacity of the pavement by:

$$C_v = (1 - x/N_{fx})^{0.33} \qquad \text{.................................... } 4.6$$

Applying the AASHTO design equation (flexible pavement case only shown here), the relationship between traffic loading capacity, N_{fx}, pavement structural number, SN, and subgrade resilient modulus, M_r, at a 50 percent reliability level and with a serviceability loss of 2.7 (initial PSI of 4.2 and terminal PSI of 1.5) is:

$$\log(N_{fx}) = 9.36\log(SN+1) - 0.2 + 2.32\log(M_r) - 8.07 \quad \ldots \; 4.7$$

Thus, a theoretical performance curve in terms of the visual condition factor, C_v, and the traffic loading, x, may be obtained from equations 4.6 and 4.7. C_v is defined over the scale of 0.0 to 1.0. ODOT rates pavement condition using a Pavement Condition Rating (PCR) index which has a scale of 1 (Very Poor) to 5 (Very Good). The following linear transformation relates these two visual condition indices:

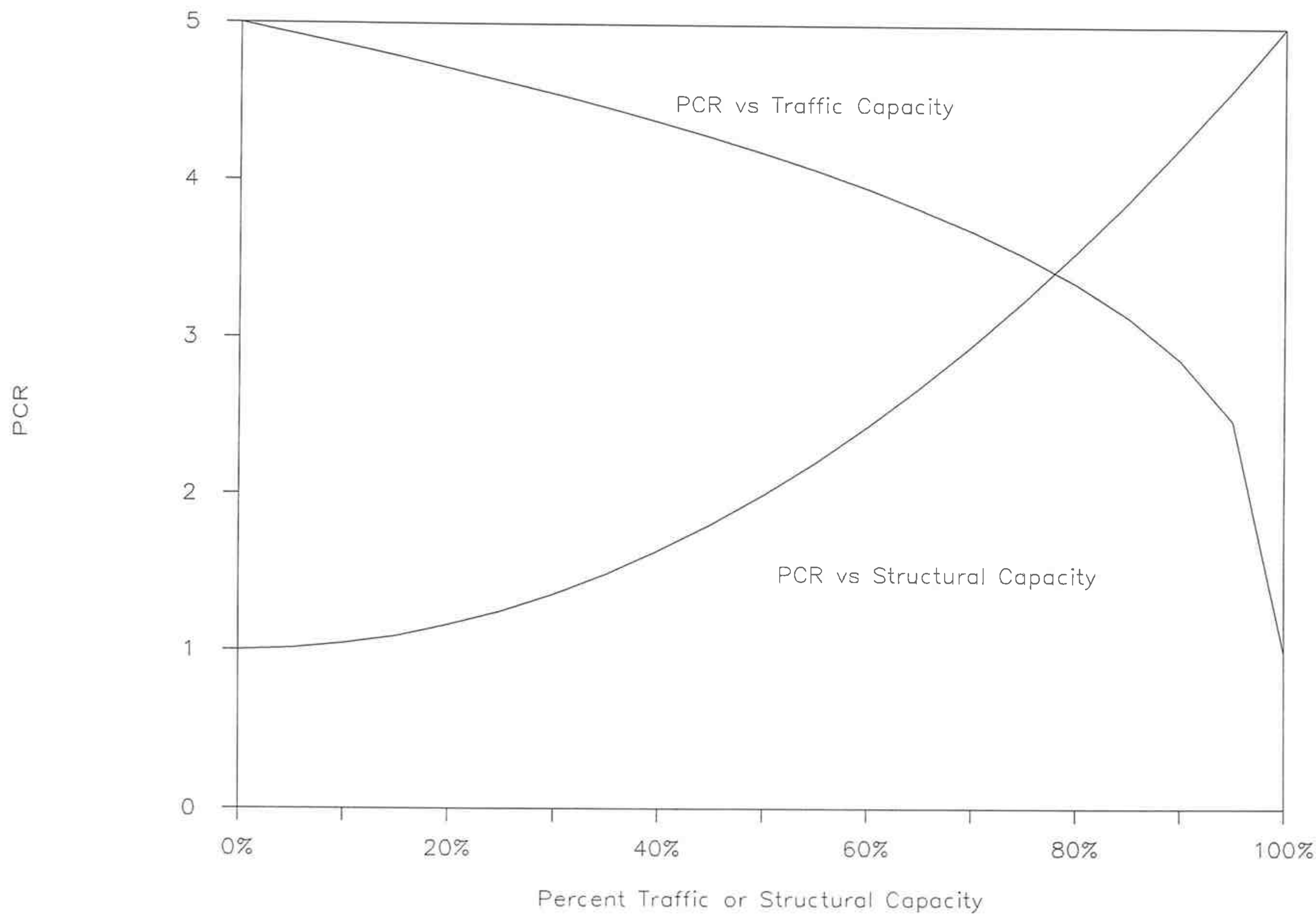
$$PCR = 1 + 4C_v \quad \ldots\ldots\ldots \; 4.8$$

The relationships between remaining life, PCR, structural capacity and traffic capacity are shown in Table 2. Since these relationships are expressed in terms of remaining life, they are independent of the pavement subgrade modulus and structural number. Also shown in Table 2 are the ODOT definitions of PCR ratings in terms of the salvage value of the pavement. Generally, pavement salvage value as estimated from the ODOT PCR corresponds with the theoretical structural condition factor as determined by remaining life. A plot of the theoretical PCR performance curve vs. percent of traffic capacity and percent of structural capacity based on the above equations is shown in Figure 5. As mentioned previously, the PCR values indicated by this performance curve refer to the PCR ratings derived from load associated distress.

Remaining Life, PCR and Pavement Capacity Relationships – Table 2

Percent Remaining Life Rlx	AASHTO Structural Condition Factor, Cx Cx = Rlx^.165	AASHTO Visual Condition Factor, Cv Cv = Rlx^.33	ODOT Pavement Condition Rating, PCR PCR = 1+4*Cv	ODOT Rating of Pavement Salvage Wear Course	Base Course
100	1.00	1.0	5.0	100%	100%
95	0.99	1.0	4.9		
90	0.98	1.0	4.9		
85	0.97	0.9	4.8		
80	0.96	0.9	4.7		
75	0.95	0.9	4.6		
70	0.94	0.9	4.6		
65	0.93	0.9	4.5		
60	0.92	0.8	4.4		
55	0.91	0.8	4.3		
50	0.89	0.8	4.2		
45	0.88	0.8	4.1		
40	0.86	0.7	4.0	80-90%	100%
35	0.84	0.7	3.8		
30	0.82	0.7	3.7		
25	0.80	0.6	3.5		
20	0.77	0.6	3.4		
15	0.73	0.5	3.1		
10	0.68	0.5	2.9	60%	100%
5	0.61	0.4	2.5		
4	0.59	0.3	2.4		
3	0.56	0.3	2.3		
2	0.52	0.3	2.1		
1	0.47	0.2	1.9	40%	60-90%
0	0.00	0.0	1.0	Combined 20-60%	

PCR vs Percent Capacity



The ODOT PCR rating categories depend primarily upon load-related distress as may be seen in Table 3. If, in actual practice, the pavements are rated according to the definitions given in Table 3, then the ODOT PCR ratings should reflect almost solely load associated distresses.

Thus, in terms of the PBA model, it should be possible to plot the observed PCR of a pavement as the "field curve" and the PCR determined by percent of applied traffic capacity as the "theoretical curve". If the field PCR ratings are significantly lower than the theoretical PCR predicated from traffic, it would indicate a significant environmental effect.

4.3 Preliminary Evaluation of Visual Condition Performance Methodology

Data Provided for Study. ODOT pavement design staff provided performance data for seven projects to test the concept of comparing the theoretical PCR curve with the field PCR curve. Data for these projects is summarized in Appendix 2. Three of the projects were located in the eastern half of the state, one of these is on the interstate system and carries a high truck traffic volume. The other two eastern projects are located on primary highways with much lower traffic loading than the interstate project. One of the four western projects is located on the interstate system and the remainder are located on primary highways. All of the projects have flexible pavement and were constructed between ten to twenty years ago.

Data that was provided for each project consisted of the following:

Project Name
Highway Name
Beginning and Ending Milepost
Year of Construction

DEFINITION OF PAVEMENT CONDITION CATEGORIES FOR INTERSTATE SYSTEM ASPHALT CONCRETE PAVEMENTS

		Salvage	
Category	Definition	Wearing Surface	Base
VERY GOOD	Stable, no cracking, no patching, no deformation. Excellent riding qualities. Usually fairly new work, but rarely more than 3 years old. No treatment would improve the roadway at this time.	100%	100%
GOOD	Stable. Very good riding qualities. Age usually 2 to 8 years. Some minor deformation evident with rutting less than ½". Distress characteristics may include: a dry or light colored appearance or minor cracking; light flushing or minor raveling.	80%-90%	100%
FAIR	Generally stable though minor areas or structural weakness evident. Riding qualities are good. Age usually 5 to 12 years old. Distress characteristics may include: deformation with rutting depths up to ¾", noticeable thermal cracks, longitudianl cracks appearing in wheel paths, or flushed with evidence of raveling.	60%	100%
POOR	Areas of instability, with marked evidence of structural deficiency. Riding qualities range from acceptable to poor. Age usually from 8 to 15 years. Distress characteristics may include: rut depths greater than ¾" or alligator cracking that requires patching. Structural requirements may range from major structural overlay to replacement of entire pavement structure. Evidence of heavy maintenance efforts.	40%	60%-90%
VERY POOR	Cost of continuously maintaining the pavement in acceptable condition may exceed available maintenance resources.	Combined WS + Base 20-60%	

Table 3 (Oregon Department of Transportation, 1987)

Asphalt Concrete Mix Class for Surface Course and Thickness
Type of Base Material and Thickness
Structural Number as Calculated by the Pavement Design Staff
Subgrade Support Value
Annual EAL Loading
Pavement Condition Ratings.

ODOT staff obtained structural information, subgrade support information and original traffic loading data from the original project files and the project construction plans. They obtained the 1990 traffic loading from the best data currently available. Since traffic data was not available for the years between construction and the present, a compound growth rate was assumed as calculated from comparison of the original and 1990 EAL volumes. The structural numbers for the pavements were calculated by the ODOT staff based on layer coefficients that are commonly used in present ODOT design practice. Several projects had lime treated subgrades and the contribution of the lime treated subgrade was ignored in the ODOT structural number calculation.

During the course of the period covered by the rating data, there were several changes in the rating system index. ODOT staff converted all ratings to a common index basis with 1 being the best and 5 being the worst rating. On four of the projects, the ODOT PCR ratings decreased during a portion of the period, i.e. the pavement condition improved. The projects in which this occurred are:

Halsey Int. - Lane County Line
Arnold Ice Caves - Horse Ridge
Brogan Hill - Brogan
Blodgett - Gelattly Summit

The Arnold Ice Caves and Brogan Hill projects received seal coats in 1988 and 1987, respectively, which explains the improvement in their condition at the rating date following the seal coat. The reasons for the improvement in condition on the other two projects is not known.

For plotting of pavement performance, a PCR index which decreases in magnitude with decreasing condition is more meaningful. Therefore, the ODOT ratings have been inverted in the plots and the data that appear in this report, except for the data in Appendix 2 which contains the ODOT-supplied data.

Development of Data. Subgrade resilient modulus values were calculated from the ODOT R-value data using the correlation equation given in the AASHTO Guide:

$$M_r = 1000 + 555*R \qquad \text{4.9}$$

where

M_r = Resilient Modulus, psi

R = Subgrade R-value

The pavement condition ratings were converted from a scale of 1 being the best state and 5 being the worst state to a scale of 1 being the worst state and 5 being the best state. Hence, the ODOT data sheets in Appendix 2 state the PCR ratings in a scale that is inverted from that used in this report.

Structural numbers were calculated for the pavements based on the following layer coefficients:

Material	AASHTO Layer Coefficient
Asphalt Conc.	0.42
Bituminous Base	0.30
Cement Treated Base	0.25
Lime Treated Subgrade	0.12
Aggregate Base	Layer coefficient based on modulus determined by:

$$M_r = K_1(\Theta)^{K_2}$$

Project	K_1	K_2	Θ
Brogan-Hill	7,000	0.5	7.5
Arnold Ice Caves	8,000	0.5	5.0
Bubbs Ranch	8,000	0.5	5.0

The bulk stress, Θ, for the aggregate base was determined from the table of values in the AASHTO Guide. A drainage coefficient of 1.0 was applied for the aggregate base.

The traffic loading capacity, N_{fx}, of the pavement was calculated using equation 4.7, above, which assumes a 50% reliability level, an initial PSI level of 4.2 and a terminal PSI level of 1.5.

The traffic growth rate during the rating period was determined by comparing the initial annual traffic loading and the 1990 estimated annual loading. This growth rate was used to calculate the cumulative traffic loading, x, at each PCR rating date.

Theoretical PCR values were calculated for each rating date using equations 4.6 and 4.8 and using both the ODOT structural numbers and the structural numbers calculated as described above. The difference between the ODOT structural number and that calculated as described above provides an indication of the range in structural number that may be obtained by different agencies using equally valid assumptions. Thus, the theoretical PCR values calculated using these two structural numbers is an indication of the envelope for the theoretical PCR value.

The computations described above are summarized in the worksheets presented in Appendix 3. These worksheets show the following data for each project:

- the two structural numbers
- traffic loading capacities
- traffic data
- field PCR ratings
- theoretical PCR values

Plots of the field PCR values and the predicted (theoretical) PCR values based on the two structural numbers are shown in Figures 6 through 12 as a function of cumulative traffic loading.

Evaluation of PCR Performance Data. The PCR values vs. cumulative traffic loading plots in Figures 6 through 12 show the following:

1) Except for the Brogan Hill and Junction City projects, the difference between the two predicted PCR curves is slight.

2) Except for the above noted projects, the difference between the two predicted (theoretical) PCR curves is not significant compared to the difference between either predicted curve and the field curve.

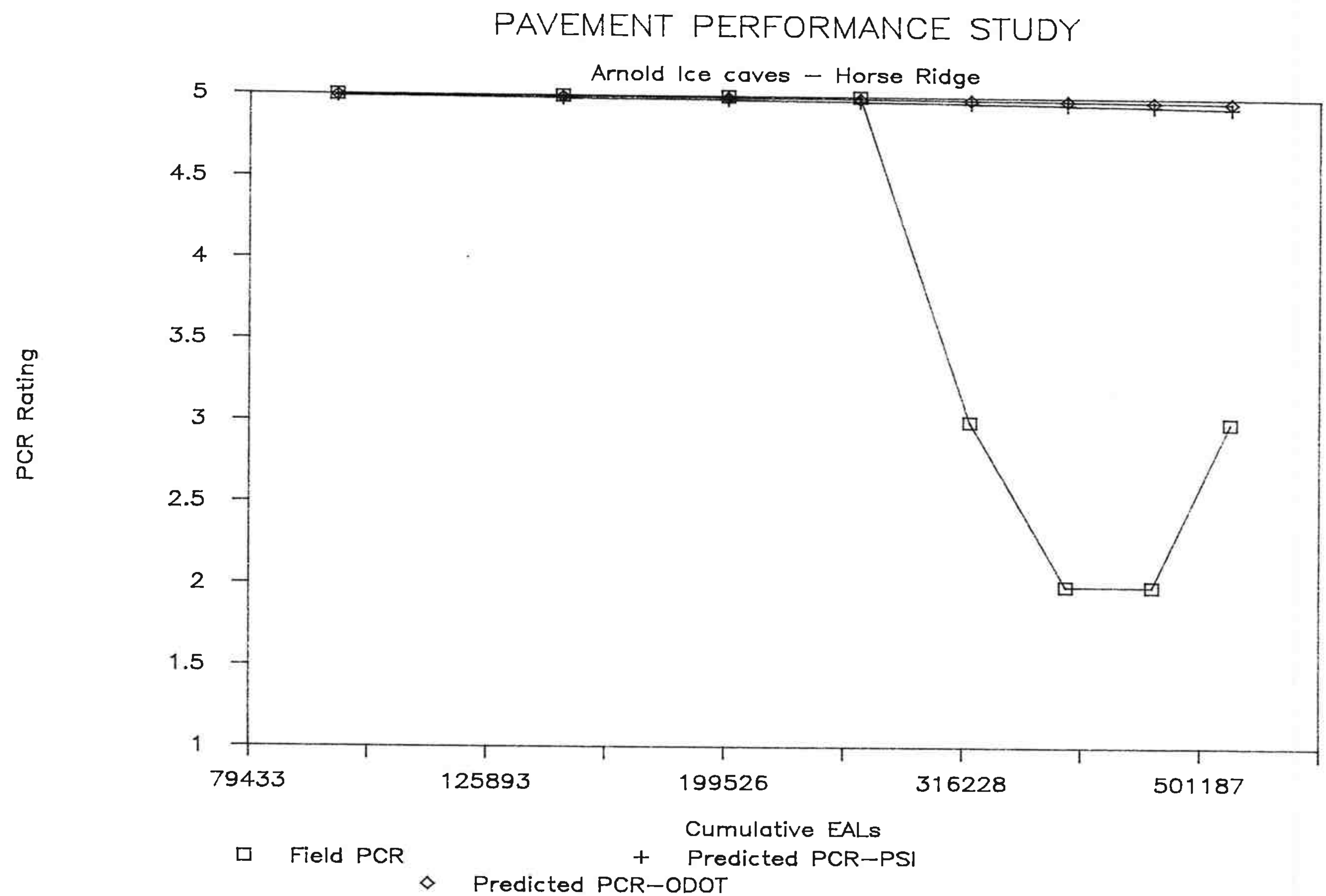
3) Except for the Brogan Hill and Junction City projects, the predicted PCR curves indicate substantial remaining life for these projects.

4) With the exception of the Junction City project, the field PCR curves all plot below, and in most cases, significantly below, the predicted PCR curve. The Junction City project includes a section of portland cement concrete pavement which apparently meanders throughout the alignment and is overlaid by 4-inches of asphalt concrete. This feature was not considered in the theoretical PCR value model for the Junction City project and this may explain why the field PCR values plot above the predicted values for the first four rating dates.

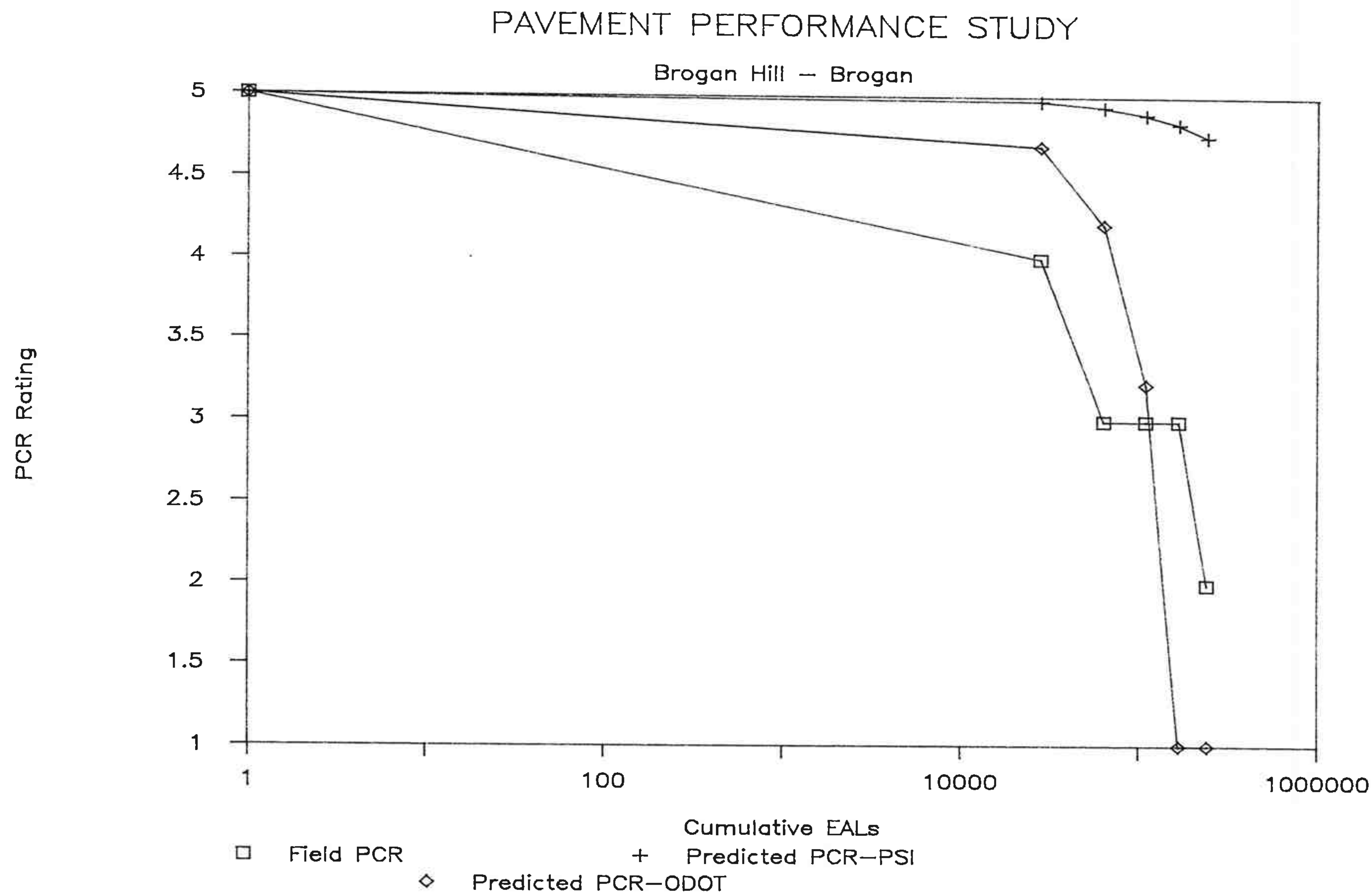
5) The magnitude of the area between the predicted and field PCR curves for Brogan Hill, Junction City, Rickreall and Blodgett projects appears to be within a range that could be reasonably attributed to environmental factors. The same is not true for the Halsey Int., Bubbs Ranch and Arnold Ice Caves projects.

6) There is no consistent relationship between the magnitude of the area between the predicted and field PCR curves and the project climate conditions.

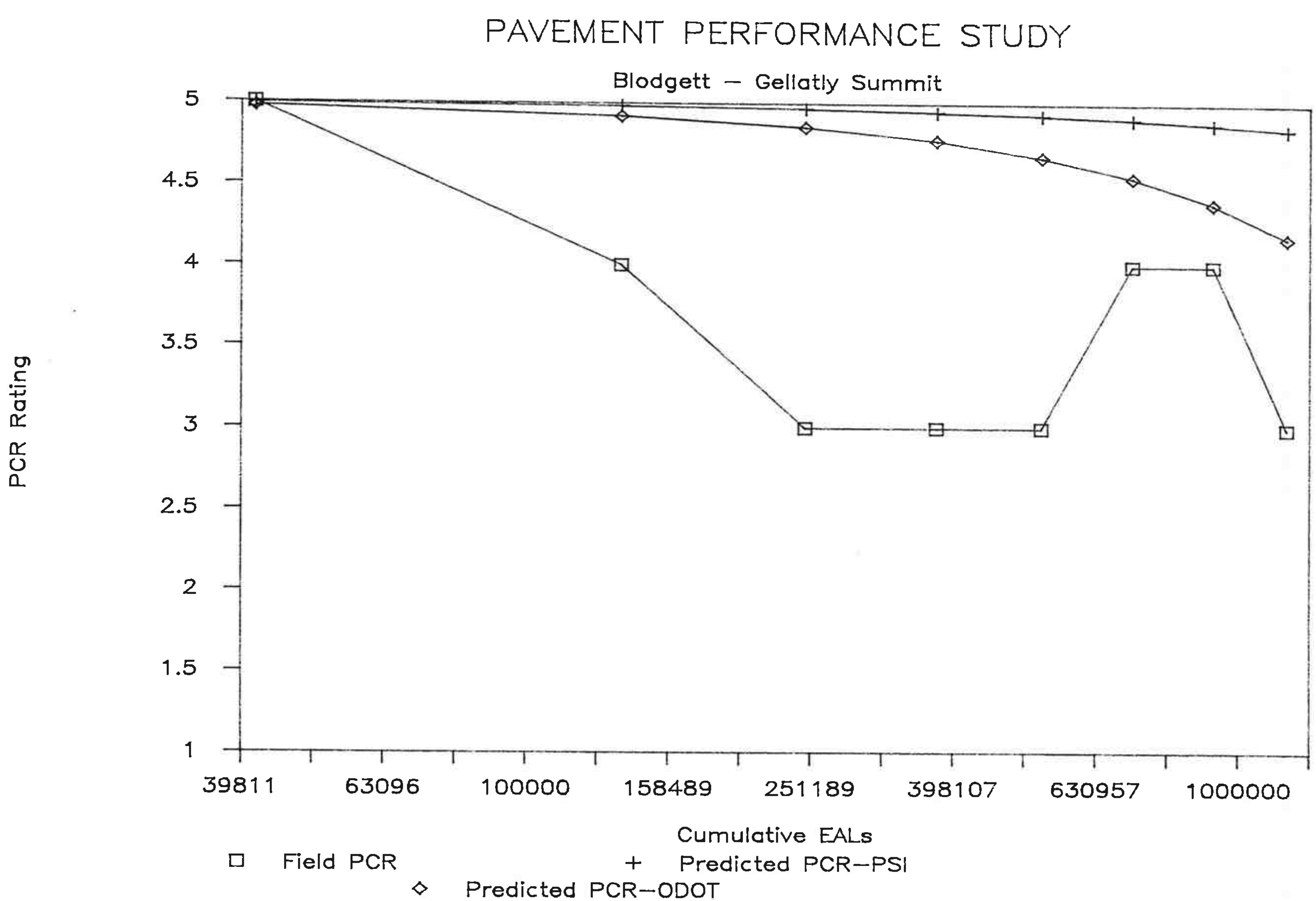
Since maintenance expenditures on these projects are unknown at present, the significance of the above observations in items 5 and 6 is difficult to judge. However, the magnitude of the area between the predicted and field PCR curves for the Halsey



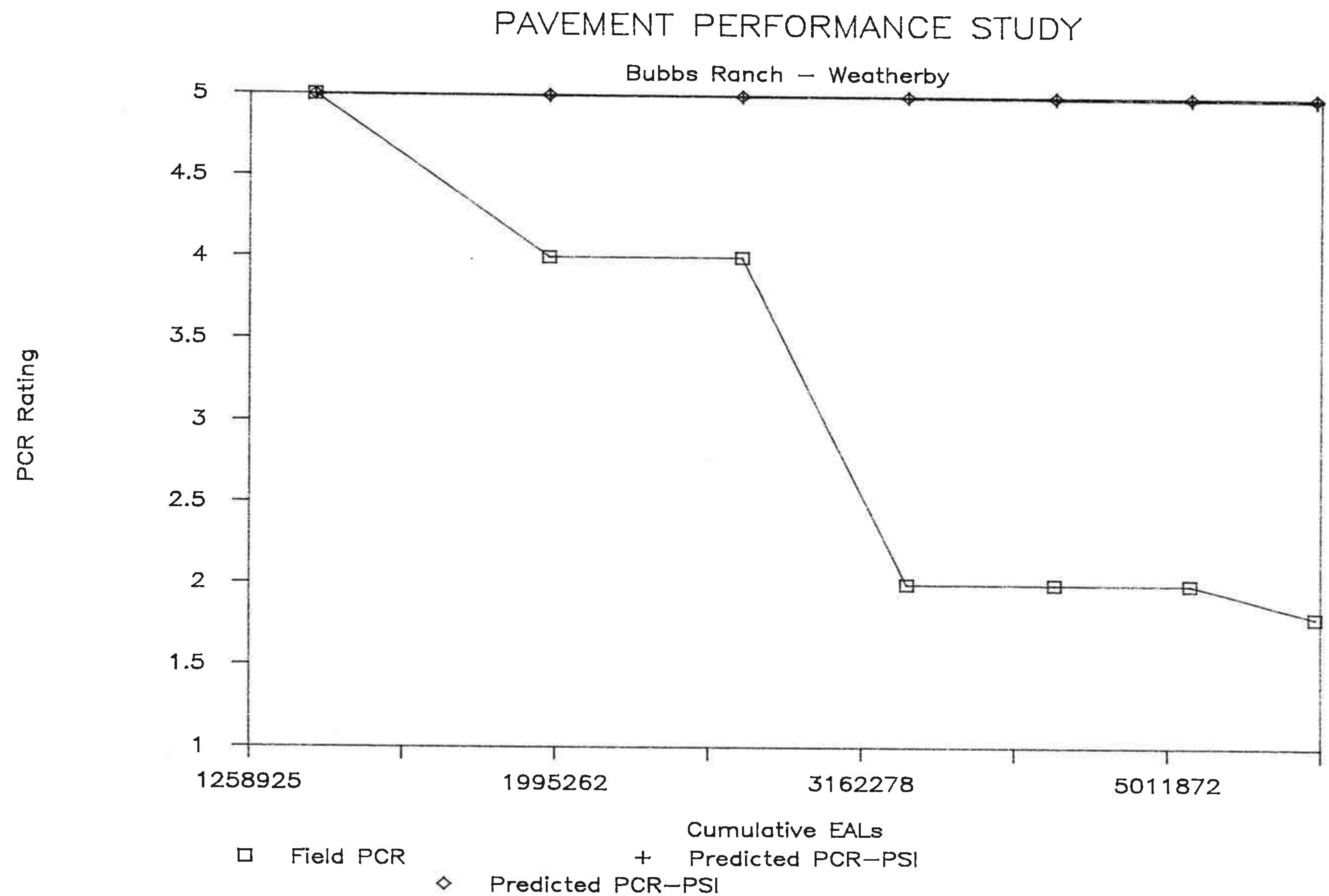
Pavement Services, Inc. Consulting Engineers for Pavements	ARNOLD ICE CAVES - HORSE RIDGE PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 6



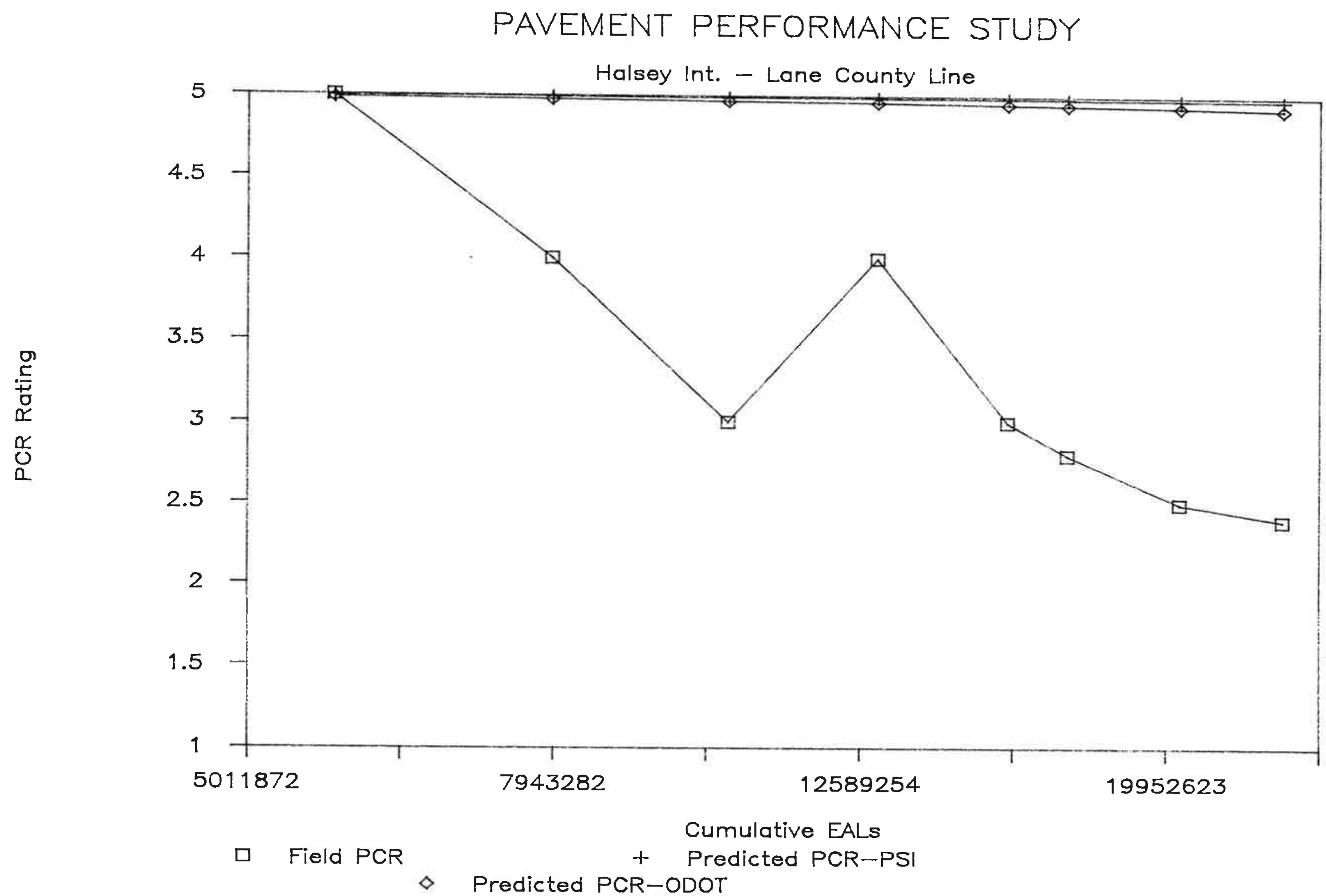
Pavement Services, Inc. Consulting Engineers for Pavements	BROGAN HILL - BROGAN PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 7



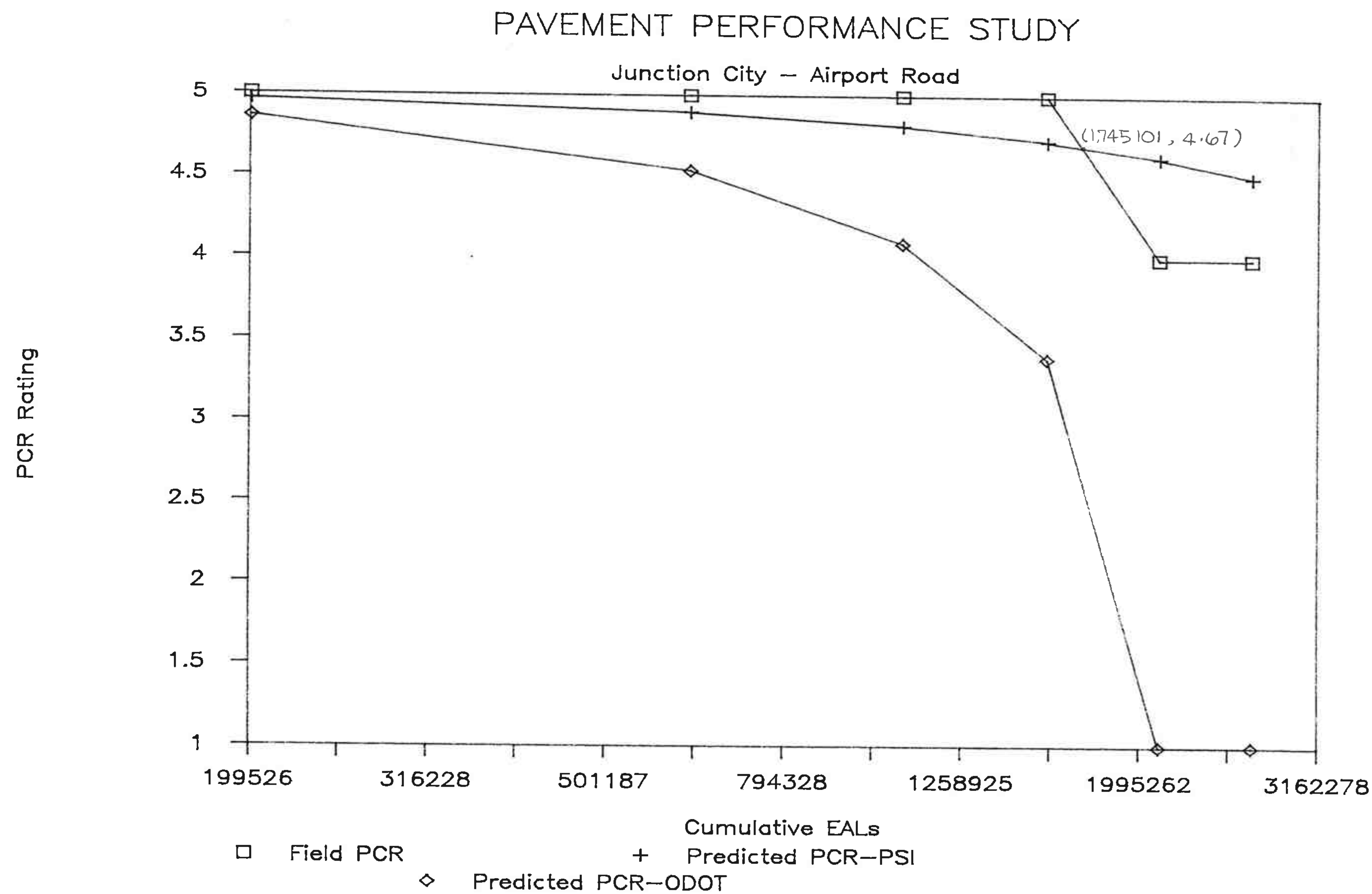
Pavement Services, Inc. Consulting Engineers for Pavements	BLODGETT - GELLATLY SUMMIT PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 8



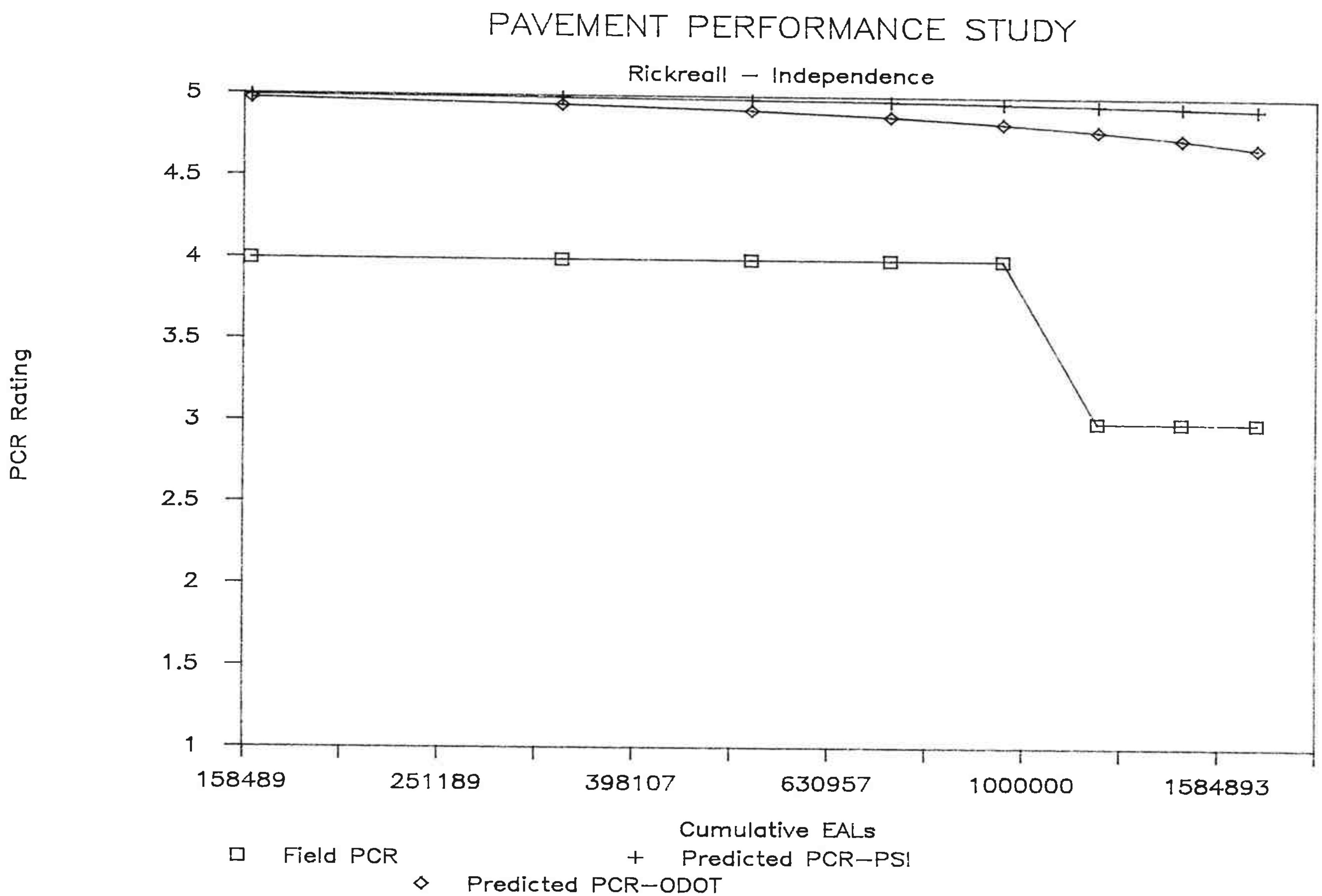
Pavement Services, Inc. Consulting Engineers for Pavements	BUBBS RANCH - WEATHERBY PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 9



Pavement Services, Inc. Consulting Engineers for Pavements	HALSEY INT. - LANE COUNTY LINE PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 10



Pavement Services, Inc. Consulting Engineers for Pavements	JUNCTIION CITY - AIRPORT ROAD PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 11



Pavement Services, Inc. Consulting Engineers for Pavements	RICKREALL - INDEPENDENCE PROJECT	VISUAL CONDITION PAVEMENT PERFORMANCE EVALUATION	FIGURE 12

Int., Bubbs Ranch and Arnold Ice Caves projects indicates that the field PCR values may reflect more than the effect of load related distress.

On the whole, the results of this evaluation appear encouraging in that the field and predicted curves are in the right relationship. Further evaluation of this data should be conducted by:

1) analysis of maintenance expenditures on these project sections to determine, or estimate, the zero-maintenance performance curve

2) inspection of the sections to verify the latest field PCR rating and to directly determine the visual condition factor, C_v

3) FWD testing to determine the subgrade modulus and effective structural capacity of the pavement section

4) obtaining ride ratings on the project section for direct estimation of their PSI

The above data would provide sufficient information to validate the concept of using a PCR based performance model for the PBA method.

4.4 Basis of General Performance Loss Prediction Model

Analysis of damage responsibilities with the PBA methodology requires that the total (zero-maintenance) performance loss be identified. This was accomplished in the Purdue study by analysis of the effect of differing levels of maintenance expenditures applied to a common section of highway. The maintenance expenditure accounting system employed by the Indiana DOT allowed for such analysis since records of maintenance expenditures were recorded by county for each highway. Thus, highways which had uniform pavement properties within several counties and to which differing levels of maintenance had been applied could be analyzed to determine the effect of maintenance on performance loss. Such maintenance data is not available in Oregon and may be difficult to estimate since maintenance cost records are kept by maintenance section which typically includes several highways. Therefore, a general performance loss prediction model which allows the effect of maintenance expenditure to be quantified in terms of variables representing pavement and environmental characteristics is desirable. Such a model would allow projects having different pavement properties and climate conditions to be jointly analyzed in order to determine the effect of maintenance.

As will be shown below, the work of Fha and Sinha in their development of predictive relationships for the proportion of load-related damage responsibility and for the maintenance effectiveness index indicates that a general performance loss predictive model is feasible.

The PBA methodology assumes that pavement deterioration may be broken down into the 4 components shown in Figure 2. These components represent the effects of load-related and non-load related factors and the effects of any interaction between the two factors. The proportions {a,b,c,d} represent the total damage due to each effect, so that:

$$a+b+c+d = 1 \qquad \text{4.10}$$

$$a+b = u \qquad \text{4.11}$$

$$c+d = e \qquad \text{4.12}$$

Hence, the model implicitly assumes that the main effects of the two factors are additive and that any interaction effects are also additive. Using the linear proportioning scheme proposed by Fha and Sinha, the relationships between {a,u,d,e} may be derived as:

$$a = 1-(1-u)^{1/2} \qquad \text{.................................... 4.13}$$

$$d = 1-(1-e)^{1/2} \qquad \text{.................................... 4.14}$$

Let (A+B) represent the total performance loss as shown in Figure 1. The loss area A is defined by the area between the no-loss line and the design equation curve, i.e., the AASHTO performance equation, and depends upon:

- pavement structural number
- cumulative traffic loading
- subgrade resilient modulus
- initial serviceability level

By definition:

$$a = A/(A+B) \qquad \text{.................................... 4.15}$$

Let $L_{i,k}$ represent the total performance loss for pavement i at age k, so that:

$$L_{i,k} = (A+B)_{i,k} \qquad \text{.................................... 4.16}$$

Thus, $L_{i,k} = A_{i,k}/a_{i,k}$ 4.17

Using the Indiana data, Fha and Sinha developed a regression relationship to predict the proportion of load related damage. They assumed a linear effects model and examined a number of variables representing pavement characteristics, traffic loading and environmental effects. They also analyzed the interaction of variables and found that interaction effects were apparently not significant. The best models that were obtained were:

a) for flexible pavement

$$Y1 = C_o + C_6X6 + C_3X3 \quad \ldots\ldots\ldots\ldots \quad 4.18$$

b) for rigid pavement

$$Y1 = C_o + C_2X2 \quad \ldots\ldots\ldots\ldots \quad 4.19$$

where

Y1 = percent responsibility of load related effects

= (a+b) = u

X6 = mean annual freezing index, degree days

X3 = structural number

X2 = pavement age, years

These relationships enable prediction of the proportion (a+b) based on pavement and environmental characteristics. From equation 4.13 and equation 4.17 the total performance loss may be predicted as shown below:

$$L_{i,k} = A_{i,k}/(1-(1-u)^{1/2}) \qquad \text{.................................. 4.20}$$

Fha and Sinha also related maintenance expenditure to pavement performance loss for a particular pavement section by the linear relationship:

$$F_{i,k} = L_{i,k} - m_{i,k}r_i \qquad \text{.................................. 4.21}$$

where

$F_{i,k}$ = Performance loss measured between the no-loss line and the field performance curve for pavement i and age k, PSI-ESAL units

$L_{i,k}$ = Total performance loss as earlier defined

$m_{i,k}$ = maintenance effectiveness, or slope of relationship between performance loss and maintenance expenditure, PSI-ESAL/lane-mile

r_i = maintenance expenditure, \$/lane-mile

Since the total loss, $L_{i,k}$, appears in both equations 4.20 and 4.21, then equation 4.21 may be written as:

$$F_{i,k} = A_{i,k}/(1-(1-u)^{1/2}) - m_{i,k}r_i \qquad \text{........................ 4.22}$$

Fwa and Sinha also analyzed maintenance effectiveness and found that it could be predicted by the model:

$$m_{i,k} = B_o + B_{12}X12 - B_2X2 \qquad \text{.......................} \; 4.23$$

where

$X12$ = soil support value

$X2$ = pavement age, years

Thus, equation 4.22 may be written as:

$$F_{i,k} = A_{i,k}/(1-(1-u)^{1/2}) - (B_o + B_{12}X12 - B_2X2)r_i \qquad \text{.....} \; 4.24$$

where

a) for flexible pavement

$$u = C_o + C_6X6 + C_3X3 \qquad \text{............................} \; 4.18$$

b) for rigid pavement

$$u = C_o + C_2X2 \qquad \text{...........................} \; 4.19$$

Although equation 4.24 is complicated, it does relate all of the parameters required to define total performance loss and, for that matter, maintenance effectiveness in terms of the measurable quantities $F_{i,k}$, $A_{i,k}$ and r_i. Thus, the model is general and removes the restriction of needing a set of homogeneous projects for analysis of the effect of maintenance.

The equations which were used to develop equation 4.24 are regression equations and undoubtedly do not represent the "true" functional relationship of the variables involved. Therefore, the important information in equation 4.24 is not the form of the equation but the identification of the key variables. Regression analysis is required to derive the parameters that apply to these variables. However, equation 4.24 is not linear in its variables and while it may be possible to perform regression on the parameters of equation 4.24, it seems better to attempt the regression assuming a simple linear effects model involving the variables identified by equation 4.24. Such a model would have the form:

$$F_{i,k} \quad = \quad K_o + K_1A_{i,k} + K_6X6 + K_{12}(X12r_i) + K_2(X2r_i) \qquad \text{......} \quad 4.25$$

The term $K_1A_{i,k}$ represents the theoretical effect of traffic loading on performance loss. Note that the variable for pavement thickness, X3, does not appear in equation 4.25. This is because the effect of pavement thickness is implicit in the computed theoretical performance loss variable, $A_{i,k}$. The term K_6X6 represents the effect of environmental conditions on performance loss. The term K_2X2r_i represents the effect of the cumulative maintenance expenditure during the life of the pavement. The term $K_{12}X12r_i$ is more difficult to interpret, but it essentially represents the performance effect of annual maintenance expenditures weighted by subgrade condition, i.e., the same maintenance expenditure applied to pavement having different subgrade conditions will have different effectiveness. The model does not include consideration of differing maintenance technology. Hence, if maintenance technology does differ significantly, then either a variable should be added to account for this factor or the data base should be stratified according to the differences in maintenance technology and that regression model should be applied separately to each stratum. The data base should also be stratified into flexible and rigid pavement types.

Once the parameters in equation 4.25 have been estimated, the zero-maintenance performance loss for a particular pavement may be calculated by the first three terms of the model, i.e.

$$L_{i,k} = K_o + K_1A_{i,k} + K_6X6 \quad \text{.........................} \quad 4.26$$

-

since the terms involving r_i disappear at zero maintenance expenditure. The proportion of damage responsibility related to "pure" load effects, a, may be estimated after determining the total performance loss, $L_{i,k}$, by application of equation 4.17. Then, the proportion of load related damage responsibility, (a+b), may be estimated by application of equation 4.13, since (a+b) = u.

The validity of the model proposed in equation 4.25 may be evaluated by the standard error and coefficient of determination that results from the regression analysis to determine the model parameters. The model could be initially tested using the seven projects that were identified for evaluation of PCR performance data. However, the regression model applied to these seven projects would have only two degrees of freedom and thus would be a very weak test.

As discussed in Section 2.4, a minimum of 18 projects should be analyzed for each pavement type. This number of projects would provide a regression model having 13 degrees of freedom which would be sufficient to test whether the model parameters are significantly different from zero.

5.0 CONCLUSIONS

5.1 Validity of PBA Method

The PBA methodology appears to be conceptually valid and there is nothing in the methodology that would make it unique to Indiana. The method does have several theoretical limitations as discussed in this report. These limitations are summarized below:

1) Certain types of traffic related distress, eg. top down cracking and studded tire wear, are not considered in the AASHTO performance equation. Hence, the performance loss due to these types of distress either will not be measured or will be combined with the performance loss due to environmental factors.

2) The AASHTO equations developed from the data collected at the Road Test do include environmental effects to some degree. However, due to the short duration of the Road Test, it is reasonable to assume that pavement deterioration due to environmental effects was very minor compared to deterioration due to load related effects. Therefore, the AASHTO performance equations can be considered to represent only load related performance.

3) The PBA method assumes that load and non-load interaction effects are proportional to the "pure" load and non-load effects. Although this assumption is logically appealing it is arbitrary. Furthermore, the results of the predictive models constructed from the Indiana study suggest that interaction effects may not actually be significant.

Whatever the ultimate form of the cost responsibility model, it will likely be, at best, only a first order approximation, eg. the present estimate that 10 percent of pavement deterioration is due to environmental factors was developed on the basis of collective judgement. In this context, the above mentioned theoretical limitations of the PBA method do not appear significant.

The adaptation of the PBA method by researchers at Oregon State University also appears conceptually valid. They extended the Purdue methodology by replacing the PSI-ESAL loss index with the more general Serviceability-ESAL Index. They also proposed using either pavement visual condition ratings or deflection measurements to define pavement performance. If deflection is used for the performance analysis, the performance loss is defined in terms of deflection-structural number units rather than serviceability-ESAL units. A procedure for calculating performance loss in serviceability units using deflection data has been presented in Section 2.2 of this report.

5.2 Suitability of Oregon's Database

Historical performance and maintenance data must be available to apply the PBA methodology. The performance and maintenance data also must cover the same section of the highway. ODOT has historical ride (Mays meter), deflection and visual condition survey data. Detailed maintenance data on specific sections of highway are available for only a limited number of highway sections. Annual accounts of maintenance expenditure are recorded by maintenance section which usually includes portions of several highways. Whether these maintenance accounts are itemized by highway is not known, although it appears doubtful that they are. Thus, to compare maintenance expenditures with performance it will be necessary to average the annual maintenance expenditures over the length of the highways included in the maintenance section. The performance data should be similarly averaged which requires that it be available for much of the highway network.

Review of the available performance data indicated that the visual condition survey data (PCR data) is the only performance data with sufficient coverage to be useful. This data has been collected on a biennial basis since 1976 and covers the entire state highway system. Additional research of maintenance records is required to determine the type and breakdown of maintenance data that is available.

5.3 Modifications to PBA Method for Oregon Implementation

To apply the PBA method with the data that is available in Oregon requires that performance be defined in terms of visual condition ratings (PCR). It will also be necessary to analyze maintenance expenditures on differing pavement sections in order to determine the effect of maintenance on performance. Both of these aspects require modification of the PBA method from that employed with the Indiana data.

It appears feasible to use PCR ratings as a measure of performance for application of the PBA method. The basis of the relationship between PCR and the AASHTO design equations is presented. Theoretical PCR performance curves were compared with field PCR performance curves for seven projects. The theoretical and field PCR values were found to have the proper relationship. Additional data on these projects is required to further analyze the validity of using PCR to define performance.

It also appears feasible to develop a general predictive relationship for performance loss in terms of variables defining pavement characteristics, environmental characteristics and maintenance expenditures. This relationship will allow the total performance loss for a particular pavement section to be predicted based on its pavement and environmental characteristics and the level of maintenance expenditure. The basis for this model is discussed and a procedure for validation of the model is suggested.

6.0 RECOMMENDATIONS

6.1 Use of PBA Method

The ability to implement the PBA method for estimation of damage responsibilities appears promising. Therefore further research in the use of the PBA method is recommended. The limitations of the method do not appear significant in comparison with alternative procedures. Procedures have been suggested for modifying the method to enable its use with the data available in Oregon. These procedures should be tested as discussed below.

6.2 Program for Development and Validation of a PBA Method

1) Research maintenance expenditure data to determine the type and breakdown of data that is available.

2) Collect field data and maintenance expenditure data on the seven projects that were identified for evaluation of the visual condition performance procedure.

3) Analyze the data collected in task 2 to:

   a) Verify the latest PCR data
   c) Estimate the visual condition factor
   b) Compare PCR values with the PCR estimated from the visual condition factor
   c) Compute performance curves using PCR, PSI and FWD data

d) Compare the above performance curves

e) Analyze the validity of using PCR to define pavement performance

4) Apply the general pavement performance loss model to the data for the seven projects.

5) If the results of task 4 are promising, then collect data for 18 flexible and rigid pavement projects and apply the general performance model to this set of data.

6) Evaluate the results of task 5 and evaluate the validity of the general performance model.

7) Develop program for implementation of the modified PBA method.

7.0 REFERENCES

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APPENDIX 1

Matching Ride & Maintenance Cost Records

ODOT 1984-1989

5/16/90

Summary

Total Maint. & Change in Mays

PAGE 1

SECTION	HWY	BEGIN MILE	END MILE	MAYS84 IPM	MAYS85 IPM	MAYS89 IPM	DIF MAYS 89-85	TOTAL 85-89 MAINTENANCE
3	1	35.69	40.83	64	63	64	1	$ 12,700
4	1	52.24	55.78	112	88	103	15	215
6	1	80.42	81.44	48	38	61	23	836
7	1	98.96	103.95	61	43	56	13	7,046
9	1	108.53	112.70	62	40	53	13	4,027
11	1	126.95	129.00	53	44	43	-1	938
12	1	143.35	146.98	69	62	66	4	1,603
14	1	197.44	203.55	38	35	44	9	0
16	1	295.33	299.00	61	60	102	42	22
17	1	300.63	305.90	90	95	92	-3	3,670
18	2	15.26	16.85	105	108	85	-23	0
19	2	42.50	47.31	57	38	57	19	0
20	2	110.73	113.90	62	51	58	7	0
21	2	149.58	159.30	40	35	33	-2	0
23	4	253.45	254.44	87	117	49	-68	0
24	4	257.86	258.80	89	116	95	-21	0
25	4	279.15	279.91	47	111	65	-46	0
28	6	216.20	218.00	46	64	62	-2	9,034
29	6	225.63	229.32	27	47	53	6	18,692
30	6	237.99	243.99	71	79	47	-32	16,953
31	6	252.97	258.96	65	60	61	1	1,987
34	7	161.37	162.30	0	67	69	2	0
35	7	176.30	177.30	0	42	63	21	0
36	7	189.32	191.96	0	139	98	-41	12,782
39	10	31.66	34.00	149	108	87	-21	26,986
40	15	12.08	14.30	0	85	66	-19	0
41	15	41.41	42.34	0	96	71	-25	948
43	19	50.78	51.73	111	103	81	-22	0
44	19	53.09	53.99	103	98	102	4	0
46	26	21.16	22.20	23	61	57	-4	0
47	26	33.55	34.50	21	63	60	-3	0
48	26	68.31	69.30	35	84	80	-4	0
49	28	55.66	56.18	202	0	0	0	0
50	30	20.95	21.70	71	0	0	0	0
51	33	23.04	24.90	0	135	113	-22	0
52	37	28.04	28.80	119	267	44	-223	0
53	42	6.09	6.99	0	95	64	-31	0
54	47	53.85	54.70	96	85	81	-4	0
55	47	62.63	67.98	48	99	65	-34	0
57	48	28.04	28.99	0	151	88	-63	0
58	52	27.73	28.50	0	177	94	-83	0
59	53	69.50	70.30	0	123	92	-31	0
60	66	11.07	12.30	75	73	107	34	0
62	91	63.91	65.99	48	39	47	8	0
64	140	27.59	29.64	0	99	84	-15	0
65	144	1.20	7.40	69	77	60	-17	0
68	162	23.34	24.30	0	126	83	-43	0
70	200	10.14	12.89	0	93	95	2	0
75	270	48.48	49.40	55	95	69	-26	0
77	456	66.24	67.30	0	74	98	24	0
78	456	86.37	87.30	0	147	111	-36	0

APPENDIX 2

Summary of Project Data for Evaluation of Visual Condition Performance Methodology

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Rickreall - Independence

HIGHWAY: Willamina - Salem No. 30

BEGINNING MP: 15.7 **END MP:** 20.6

YEAR CONSTRUCTED: 1972

PAVEMENT TYPE: "B" AC **PAVEMENT DEPTH:** 4"

BASE TYPE: Cement Treated Base **BASE DEPTH:** 9"
Lime Treated Subgrade 6"

EQUIVALENT STRUCTURAL NUMBER: 3.84
(No credit given for L.T.S.)

SUBGRADE SUPPORT VALUE: R Value = 9

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1974	79,053
1990	149,345

PAVEMENT CONDITION RATING:

1976	2
1978	2
1980	2
1982	2
1984	2
1986	3
1988	3
1990	3

COMMENTS:

(PCR4W.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Blodgett - Gellatly Summit

HIGHWAY: Corvallis - Newport No. 33

BEGINNING MP: 39.4 **END MP:** 41.2

YEAR CONSTRUCTED: 1975

PAVEMENT TYPE: "B" or "C" AC **PAVEMENT DEPTH:** 4"

BASE TYPE: Cement Treated Base **BASE DEPTH:** 8"
Lime Treated Subgrade 6"

EQUIVALENT STRUCTURAL NUMBER: 3.60
(No credit given for L.T.S.)

SUBGRADE SUPPORT VALUE: R Value = 6

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1975	41,673
1990	139,839

PAVEMENT CONDITION RATING:

1976	1
1978	2
1980	3
1982	3
1984	3
1986	2
1988	2
1990	3

COMMENTS: A nearby project, MP 44.00 - 44.50 Passing Lanes, had a subgrade resilient modulus test of 7,000 psi.

Our records offer no ready explanation of the improved condition ratings in 1986 and 1988.

(PCR6W.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Halsey Int. - Lane County Line

HIGHWAY: Pacific No. 1

BEGINNING MP: 203.6 **END MP:** 216.2

YEAR CONSTRUCTED: 1970

PAVEMENT TYPE: "B" AC **PAVEMENT DEPTH:** 19"

BASE TYPE: N/A **BASE DEPTH:** N/A

EQUIVALENT STRUCTURAL NUMBER: 6.78

SUBGRADE SUPPORT VALUE: R Value = 7

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1970	860,462
1990	1,877,117

PAVEMENT CONDITION RATING:

1976	1
1978	2
1980	3
1982	2
1984	3
1985	3.2
1987	3.5
1989	3.6

COMMENTS: Existing section appears to be full depth AC.

The records indicate that a re-evaluation was done on this section in 1982, perhaps thinking that the previous rating may not have accurately reflected the pavement condition.

(PCR1W.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Bubbs Ranch - Weatherby

HIGHWAY: Old Oregon Trail No. 6

BEGINNING MP: 327.1 **END MP:** 336.2

YEAR CONSTRUCTED: 1973

PAVEMENT TYPE: "B" AC **PAVEMENT DEPTH:** 6"

BASE TYPE:	**BASE DEPTH:**
Plant Mix Bit. Base	5"
Aggregate Base	4"
Select Subgrade	18"

EQUIVALENT STRUCTURAL NUMBER: 5.56

SUBGRADE SUPPORT VALUE: R Value = 38

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1970	197,892
1990	664,237

PAVEMENT CONDITION RATING:

1976	1
1978	2
1980	2
1982	4
1984	4
1985	4.0
1987	4.0
1989	4.2

COMMENTS:

(PCR4E.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Junction City - Airport Road

HIGHWAY: Pacific West No. 1W

BEGINNING MP: 109.7 **END MP:** 116.7

YEAR CONSTRUCTED: 1979

PAVEMENT TYPE: Class "B" AC **PAVEMENT DEPTH:** 4"

BASE TYPE: Cement Treated Base **BASE DEPTH:** 10"
Lime Treated Subgrade 6"

EQUIVALENT STRUCTURAL NUMBER: 4.08
(No credit given for L.T.S.)

SUBGRADE SUPPORT VALUE: R Value = 3

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1980	207,767
1990	299,817

PAVEMENT CONDITION RATING:

1978	5
1980	1
1982	1
1984	1
1986	1
1988	2
1990	2

COMMENTS: The final plans indicate there is a 22' wide slab of pre-existing PCC under part of this project. It meanders, sometimes being under the southbound lanes, and sometimes under the northbound lanes. The full four inches of AC was placed over this concrete.

(PCR2W.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Arnold Ice Caves - Horse Ridge

HIGHWAY: Central Oregon No. 7

BEGINNING MP: 12.4 **END MP:** 18.8

YEAR CONSTRUCTED: 1972

PAVEMENT TYPE: "B" AC **PAVEMENT DEPTH:** 5.5"

BASE TYPE: Plant Mix Bit. Base **BASE DEPTH:** 3"
Aggregate Base 12"

EQUIVALENT STRUCTURAL NUMBER: 4.65

SUBGRADE SUPPORT VALUE: R Value = 8

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1972	22,442
1990	39,013

PAVEMENT CONDITION RATING:

1976	1
1978	1
1980	1
1982	1
1984	3
1986	4
1988	4
1990	3

COMMENTS: Our records indicate that this section received a sealcoat in 1988. This would explain the improved Pavement Condition Rating in 1990.

(PCR4E.MEM)

OREGON STATE HIGHWAY DIVISION

PAVEMENT DESIGN GROUP

PAVEMENT PERFORMANCE STUDY

June, 1990

PROJECT NAME: Brogan Hill - Brogan

HIGHWAY: John Day No. 5

BEGINNING MP: 246.7 **END MP:** 254.6

YEAR CONSTRUCTED: 1980

PAVEMENT TYPE: "B" AC **PAVEMENT DEPTH:** 3"

BASE TYPE: Aggregate Base **BASE DEPTH:** 2"
Aggregate Subbase 12"

EQUIVALENT STRUCTURAL NUMBER: 2.20

SUBGRADE SUPPORT VALUE: R Value = 8

SOURCE OF SUBGRADE SUPPORT VALUE: This project.

TRAFFIC (ANNUAL EAL'S):

1980	13,150
1990	43,828

PAVEMENT CONDITION RATING:

1976	3
1978	4
1980	1
1982	2
1984	3
1986	3
1988	2
1990	4

COMMENTS: Our records indicate that this section received a seal coat in 1987. This would explain the improved condition rating in 1988.

(PCR8E.MEM)

APPENDIX 3

Visual Performance Evaluation Worksheets

PAVEMENT PERFORMANCE STUDY

PROJECT:	Arnold Ice Caves - Horse Ridge	
HIGHWAY:	Central Oregon No. 7	
YEAR CONSTRUCTED:	1972	1972
STRUCTURAL NUMBER:	4.19	4.65
SUBGRADE Mr:	5,400	5,400
Nfx:	12,109,327	26,811,863

TRAFFIC DATA:

Year	EAL's	Growth Rate:	3.1%
1972	22442		
1990	39013		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1976	94,057	5.0	5.0	1%	0	5.0	1%
1978	145,601	5.0	5.0	1%	0	5.0	1%
1980	200,411	5.0	5.0	2%	0	5.0	2%
1982	258,695	5.0	5.0	2%	60004	5.0	2%
1984	320,672	3.0	5.0	3%	162194	5.0	3%
1986	386,576	2.0	5.0	3%	206981	5.0	3%
1988	456,656	2.0	4.9	4%	182234	5.0	4%
1990	531,177	3.0	4.9	4%		5.0	4%
				Total	611,413		

PAVEMENT PERFORMANCE STUDY

PROJECT:	Blodgett - Gellatly Summit	
HIGHWAY:	Corvallis - Newport No.33	
YEAR CONSTRUCTED:	1975	1975
STRUCTURAL NUMBER:	4.4	3.6
SUBGRADE Mr:	4,300	4,300
Nfx:	10,347,904	2,307,067

TRAFFIC DATA:

Year	EAL's	Growth Rate:	8.4%
1975	41673		
1990	139839		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1976	41,673	5.0	5.0	0%	46005	5.0	0%
1978	135,822	4.0	5.0	1%	163246	4.9	1%
1980	246,464	3.0	5.0	2%	254826	4.9	2%
1982	376,487	3.0	5.0	4%	296637	4.8	4%
1984	529,288	3.0	4.9	5%	254870	4.7	5%
1986	708,855	4.0	4.9	7%	188486	4.5	7%
1988	919,879	4.0	4.9	9%	337761	4.4	9%
1990	1,167,869	3.0	4.8	11%		4.2	11%
				Total	1,541,832		

PAVEMENT PERFORMANCE STUDY

PROJECT:	Brogan Hill - Brogan	
HIGHWAY:	John Day No. 5	
YEAR CONSTRUCTED:	1980	1980
STRUCTURAL NUMBER:	3.12	2.2
SUBGRADE Mr:	5,400	5,400
Nfx:	1,395,055	131,026

TRAFFIC DATA:

Year	EAL's	Growth Rate:	12.8%
1980	13150		
1990	43828		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1980	1	5.0	5.0	0%	13618	5.0	0%
1982	27,982	4.0	5.0	2%	51838	4.7	2%
1984	63,582	3.0	4.9	5%	86802	4.2	5%
1986	108,873	3.0	4.9	8%	107459	3.2	8%
1988	166,494	3.0	4.8	12%	168398	1.0	12%
1990	239,801	2.0	4.8	17%		1.0	17%
				Total	428,114		

PAVEMENT PERFORMANCE STUDY

PROJECT:	Brogan Hill - Brogan	
HIGHWAY:	John Day No. 5	
YEAR CONSTRUCTED:	1980	1980
STRUCTURAL NUMBER:	3.12	2.2
SUBGRADE Mr:	5,400	5,400
Nfx:	1,395,055	131,026

TRAFFIC DATA:

Year	EAL's	Growth Rate:	12.8%
1980	13150		
1990	43828		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1980	1	5.0	5.0	0%	13618	5.0	0%
1982	27,982	4.0	5.0	2%	51838	4.7	2%
1984	63,582	3.0	4.9	5%	86802	4.2	5%
1986	108,873	3.0	4.9	8%	107459	3.2	8%
1988	166,494	3.0	4.8	12%	168398	1.0	12%
1990	239,801	2.0	4.8	17%		1.0	17%
				Total	428,114		

PAVEMENT PERFORMANCE STUDY

PROJECT:	Halsey Int. - Lane County line	
HIGHWAY:	Pacific No. 1	
YEAR CONSTRUCTED:	1970	1970
STRUCTURAL NUMBER:	7.98	6.78
SUBGRADE Mr:	4,900	4,900
Nfx:	1,636,520,376	427,389,584

TRAFFIC DATA:

Year	EAL's	Growth Rate:	4.0%
1970	860462		
1990	1877117		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1976	5,704,149	5.0	5.0	0%	1096742	5.0	0%
1978	7,922,044	4.0	5.0	0%	3579060	5.0	0%
1980	10,319,867	3.0	5.0	1%	3864173	5.0	1%
1982	12,912,214	4.0	5.0	1%	4171523	5.0	1%
1984	15,714,865	3.0	5.0	1%	3099698	5.0	1%
1985	17,200,334	2.8	5.0	1%	7355837	4.9	1%
1987	20,350,859	2.5	5.0	1%	8624719	4.9	1%
1989	23,756,971	2.4	5.0	1%		4.9	1%

Total 31,791,753

PAVEMENT PERFORMANCE STUDY

PROJECT:	Junction City - Airport Road	
HIGHWAY:	Pacific West No. 1W	
YEAR CONSTRUCTED:	1979	1979
STRUCTURAL NUMBER:	4.9	4.08
SUBGRADE Mr:	2,700	2,700
Nfx:	8,052,649	1,984,487

TRAFFIC DATA:

Year	EAL's	Growth Rate:	3.7%
1979	200285		
1980	207767		
1990	299817		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area
1980	200,285	5.0	5.0	2%	
1982	623,580	5.0	4.9	8%	
1984	1,079,092	5.0	4.8	13%	
1986	1,569,272	5.0	4.7	19%	109196
1988	2,096,759	4.0	4.6	26%	319106
1990	2,664,391	4.0	4.5	33%	
				Total	428,302

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx
1980	200,285	5.0	4.9	2%
1982	623,580	5.0	4.5	8%
1984	1,079,092	5.0	4.1	13%
1986	1,569,272	5.0	3.4	19%
1988	2,096,759	4.0	1.0	26%
1990	2,664,391	4.0	1.0	33%

PAVEMENT PERFORMANCE STUDY

PROJECT:	Rickreall - Independence	
HIGHWAY:	Willamina - Salem No. 30	
YEAR CONSTRUCTED:	1972	1972
STRUCTURAL NUMBER:	4.65	3.84
SUBGRADE Mr:	6,000	6,000
Nfx:	34,236,107	8,043,841

TRAFFIC DATA:

Year	EAL's	Growth Rate:	4.1%
1974	79053		
1990	149345		

Year	Cumulative EALs	Field PCR	Predicted PCR	x/Nfx	Area	Predicted PCR	x/Nfx
1976	161,312	4.0	5.0	0%	172984	5.0	0%
1978	335,976	4.0	5.0	1%	185966	4.9	1%
1980	525,095	4.0	5.0	2%	199786	4.9	2%
1982	729,867	4.0	5.0	2%	214471	4.9	2%
1984	951,586	4.0	5.0	3%	350079	4.8	3%
1986	1,191,656	3.0	5.0	3%	506456	4.8	3%
1988	1,451,595	3.0	4.9	4%	545348	4.7	4%
1990	1,733,048	3.0	4.9	5%		4.7	5%
				Total	2,175,091		