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Pavement Management and Weigh-in-Motion

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Contents

Foreword

This Record consists of 15 papers, of which 10 are on pavement management concepts and techniques and 5 are on heavy vehicle classification, mainly weigh-in-motion systems. Of the papers on pavement management, 2 are on systems undertaken for local government and 6 are on the topic of pavement condition (or performance) and its relation to maintenance needs. Presented in the remaining 2 papers on pavement management issues are a discussion on the development of a framework and computer program for pavement management, and a roadway information system adapted to an interactive graphics system used by the utility industry. Of the papers on weigh-in-motion, research is presented in several on the calibration and accuracy of weigh-in-motion systems for heavy vehicles. The results of a feasibility study on automatically monitoring truck tire pressures are discussed in another paper.

Cation et al. document the development of a preventive maintenance algorithm and introduce a new concept in determining distress density limits for the recommendation of preventive maintenance treatments.

Butt et al. describe a pavement performance and prediction model that has been developed based on the pavement condition index and the age of the pavement. The life span of the pavement is divided into zones, with each zone representing a period of 6 years. If the state of any given pavement section is known, its future condition can be predicted efficiently from the corresponding transition matrices. The model presented in this paper might play an integral part in the decision-making procedure for determining optimal maintenance and repair strategies.

Described in the paper by Bottiger and Kilareski is a research project that used the utility industry's interactive graphic data base for highway applications. As a roadway section is similar to an electric line, many of the modeling concepts are also similar. A computer model of the highway system was developed that used five types of facilities and a case study was conducted to test the model.

Sharaf et al. present a network level procedure for determining the best maintenance and repair alternative and its associated cost for different pavement categories at different pavement condition index ranges. Data from a number of military installations in the United States were used and the analysis was performed separately for each installation.

The applicability of three mathematical curve-fitting techniques for modeling pavement condition deterioration behavior is presented in the paper by Shahin et al. The mathematical models investigated are stepwise regression, B-spline approximation, and constrained least-squares estimation. The best features of each are integrated into an interactive format capable of operating within the PAVER pavement management system. Procedures used are said to constitute a complete method to accurately model and predict pavement family behavior and pavement section behavior. The method is being integrated into the PAVER pavement management system to improve the pavement evaluation process.

Monismith et al. describe the results of a study of pavement management practices used by 13 local government agencies in the United States and Canada. Included is a discussion of factors to be considered in planning and developing a pavement management system based on the experiences of these organizations. In the planning phase, consideration is given to resource and information requirements, and specific considerations associated with actual development are cited. Practices are summarized in a series of tables for ready reference.

An approach to allocation of funds for maintenance and rehabilitation of pavements has been developed by Smith et al. The approach uses a minimum of information to make reasonable budget analyses on maintenance and rehabilitation needs with unconstrained funding. Described in this paper is the way in which funding needs are allocated when funding is less than needs. This includes consideration of the condition of the pavement, change of condition over time, cost of the maintenance or rehabilitation over time, and stop-gap maintenance generated by

deferring maintenance. This process was accomplished by making it simple for public works personnel to visualize and use. It is part of a network-level microcomputer-based pavement management system developed for San Francisco Bay Area agencies.

Flanagan and Halbach discuss expert systems as a part of pavement management. Their structures are defined and compared with current pavement management systems. Ways in which they can enhance pavement management systems are examined, as are their current limitations.

Hudson et al. describe a computerized pavement data analysis methodology called MAPCON. Pavement condition data is input and output results are said to be useful for pavement management. The type of data analyzed by MAPCON includes friction and skid, roughness, structural capacity, surface condition, or a combination of the last three. MAPCON is available for implementation and use by highway agencies, although continued support for further research and development of MAPCON is considered desirable.

Presented in the paper by Luhr and Fernando is the development of rational guidelines for the posting of axle load limits in Pennsylvania. A performance model based on the present serviceability index was developed that relates pavement performance to subgrade strain. A procedure was devised that determines the subgrade strain from measurements taken with either the Road Rater or the falling weight deflectometer. A microcomputer program was set up to generate information concerning predicted years to failure for different axle load limits. In addition, simple charts were developed to allow engineers to conduct a load-limit analysis in the absence of deflection measurements, and to determine pavement damage responsibility for different axle loads.

Bell and Krukar present part of a comprehensive automatic traffic data collection made up of weigh-in-motion, automatic vehicle classification, and automatic vehicle identification systems. Weigh-in-motion determines axle and vehicle weight at full speed on the highway, automatic vehicle classification arranges the traffic into groups by identification of axle spacings, and automatic vehicle identification acts as an electronic license plate, used with weigh-in-motion and automatic vehicle classification to characterize individual vehicles.

Presented in the paper by Cunagin and Grubbs are the results of a study on the feasibility of automatically monitoring the contact tire pressures produced by trucks while they are in motion by keeping track of tire footprint dimensions and weight.

Davies and Sommerville examine the problems of calibration and accuracy testing for weigh-in-motion systems, and propose several approaches that might improve the present situation. Existing calibration techniques are described and a different approach advocated. A statistical appraisal of the new approach is made. A new technique is also described for self-calibration of weigh-in-motion systems. It is suggested that self-calibration offers the prospect of improved weigh-in-motion accuracy in between conventional calibration exercises.

A study by Izadmehr and Lee of in-motion weighing of some 800 trucks selected from the traffic stream on I-10 near Seguin, Texas, yielded data sets that were analyzed to define the attainable accuracy within which wheel, axle, axle-group, and gross-vehicle weights could be estimated by a properly calibrated in-pavement weigh-in-motion system. The concept of use tolerances, which allow for probable error in both the static weight measurement and weigh-in-motion weight estimate, is presented. Tolerances for high, intermediate, and low weigh-in-motion scales at the experimental site are tabulated.

A second paper by Izadmehr and Lee illustrates the importance of on-site calibration for weigh-in-motion systems by comparing weigh-in-motion weight estimates with corresponding wheel weights measured on a special static reference scale. Various truck types were included in the analysis, and high, intermediate, and low speeds of in-motion weighing were considered. The variability in weigh-in-motion weight estimates was not affected appreciably by the type of moving-vehicle loading that was used as the basis for calibration. Suggestions are offered concerning the types of trucks and the minimum number of wheel loads that should be used as the basis for on-site calibration.

Development of a Preventive Maintenance Algorithm for Use in Pavement Management Systems

KATHRYN A. CATION, MOHAMED Y. SHAHIN, THOMAS SCULLION, AND ROBERT L. LYTTON

A primary objective of Pavement Management Systems is to maintain pavements in good condition at the lowest possible cost. Preventive maintenance treatments play an important role in prolonging service life by slowing down pavement deterioration. This paper documents the development of a preventive maintenance algorithm and introduces a new concept in determining distress density limits for the recommendation of preventive maintenance treatments. A literature search was performed in order to evaluate preventive maintenance algorithms currently in use. Common to existing preventive maintenance algorithms is the use of the subjective judgment of pavement engineers to determine distress density ranges. The procedure described in this paper, which was developed at the United States Army Construction Engineering Research Laboratory, relates distress density directly to the Pavement Condition Index used in the PAVER Pavement Management System. The concepts presented in this paper can be used by any agency to fully develop a preventive maintenance algorithm. The procedure described can be applied to both asphalt concrete and portland cement concrete pavements, and is flexible enough to allow for local policies and economic factors. The initial algorithm may be expanded to include environmental or geographic factors and additional preventive maintenance treatments at a later date.

As the infrastructure of pavements in the United States continues to deteriorate, many agencies are using Pavement Management Systems (PMS) as an aid in maintaining pavements in good condition at the lowest possible cost. An effective PMS should include

1. Data storage and report generation;
2. Network management tools, including prediction of pavement condition, budget planning, and inspection scheduling; and
3. Project management tools, including pavement condition history, construction history, and economic analysis for determining the most cost-effective maintenance and repair (M&R) strategy.

The timing of M&R repairs can be an important factor in maintaining pavements economically. Typical pavement deterioration curves depict pavement life cycles as consisting of two phases *(1)*. During the first phase, a 40 percent deterioration of pavement condition gradually occurs over 75 percent of the life of the pavement. As the second phase begins, a sharp decrease in condition occurs. An equivalent 40 percent drop in condition takes place within only 12 percent of the life of the pavement. Pavement M&R costs at this point are 4 to 5 times as high as those at the end of the first phase. If pavement repairs are performed while the pavement is still in the first phase, rather than waiting until the sharp decline to a poor or failed condition, costs can be greatly reduced. A method of deterring the onslaught of the sharp decrease in condition is to perform appropriate preventive maintenance techniques to pavements relatively free of surface distresses. The function of these preventive maintenance techniques is to slow down pavement deterioration in order to prolong service life.

The objective of this paper is to document the development of a preventive maintenance algorithm for use in Pavement Management Systems. The PAVER Pavement Management System, developed by the U.S. Army Construction Engineering Research Laboratory (USA-CERL), will be used to demonstrate the applicability of the algorithm. However, the logic followed throughout the development is equally applicable to any PMS. This paper will also introduce a new concept in determining distress density limits for identifying appropriate preventive maintenance strategies. The concept presented is an improvement over existing subjective approaches.

DEVELOPMENT OF THE PAVEMENT CONDITION INDEX (PCI)

In order to predict future pavement conditions, a Pavement Management System must have an objective, repeatable measurement rating. The PAVER System is based on the Pavement Condition Index (PCI). The development of the PCI is well documented *(2)*. It is important to explain the concepts behind the PCI for an understanding of the components of the preventive maintenance algorithm presented later in this paper. The PCI will be used in the determination of which pavements should be recommended for preventive maintenance, and one of the steps used in calculating the PCI is fundamental to a major portion of the preventive maintenance algorithm.

There were three objectives to be met in the development of the PCI. It was meant to provide: *(a)* an index of present condition in terms of both structural integrity and surface

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operational condition, *(b)* an objective, rational basis for determining M&R needs and priorities, and *(c)* a warning system for the early identification or projection of major repair requirements or both *(3)*. The PCI is based on three pavement distress characteristics: distress type, severity, and quantity. These three characteristics are evaluated according to a standardized rating system, and the PCI, a numerical condition index between 0 and 100, is determined.

Because of the large number of possible distress type/severity/quantity combinations that are possible, the major problem was the development of one index that would take into account all three factors. The following equation was found to be a comprehensive and accurate model for expressing a condition index *(3)*.

$$PCI = C - \sum_{i=1}^{p} \sum_{j=1}^{m_i} a\,[\,T_i, S_j, D_{ij}]\,F\,(t, d)$$

where

- PCI = pavement condition index;
- C = a constant depending on desired maximum scale value;
- a [ ] = deduct weighting value depending on distress type T_i, level of severity S_j, and density of distress D_{ij};
- i = counter for distress types;
- j = counter for severity levels;
- p = total number of distress types of pavement type under consideration;
- m_i = number of severity levels on the ith type of distress; and
- $F(t, d)$ = an adjustment factor for multiple distresses that varies with total summed deduct value (t) and number of deducts (d).

Acceptable distress definitions and deduct values were developed over several years through extensive field testing and revisions by a group of experienced pavement engineers. During field testing, a subjective pavement condition rating (PCR) was determined for each section of pavement in addition to a calculated PCI. In order to calculate the PCI, deduct values were preliminarily assigned to all distress type/severity level combinations based on distress density [(amount of distress/area of sample unit) × 100]. The deduct values were meant to serve as a type of weighting factor that indicated the size of the effect that the particular distress type/severity level/distress density combination had on the pavement condition. The deduct curves for alligator cracking on roads and streets are shown in Figure 1.

Once all the deduct values had been determined for each distress type/severity level combination identified in the pavement survey, the total number of deducts was summed. The total deduct value was then corrected based on the number of deducts and subtracted from 100, which was chosen to be representative of a pavement in perfect condition. Many revisions were made until the calculated PCI could closely correlate with the average PCR value.

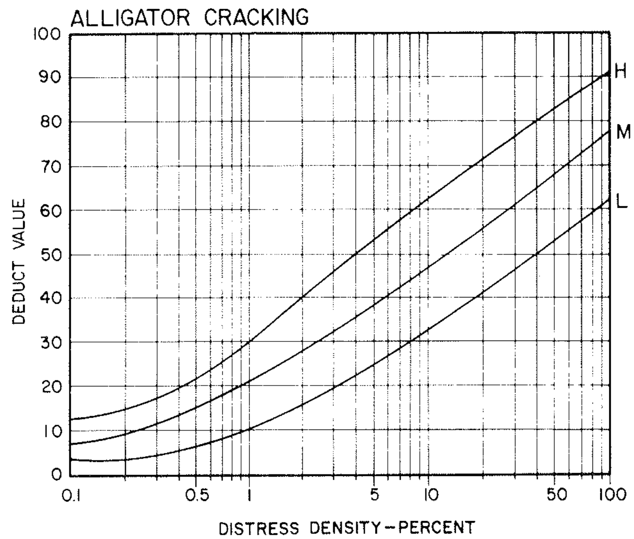

FIGURE 1 Deduct value curves for alligator cracking.

Based on the input from the field testing and evaluation procedure, accurate descriptions of distress types, severity levels, and the corresponding deduct values were derived so that a composite distress index (PCI) could be determined. The continued ability of the PCI to represent the subjective rating of pavement engineers was recently confirmed in a study done in the San Francisco Bay area *(4)*. In that study, inspections using both subjective ratings and the PCI rating procedures were performed on Bay area pavements to test the accuracy of the deduct and multiple-distress correction curves. Results of the regression analysis on the collected data indicated a high correlation between the mean subjective rating (PCR) and PCI ($R^2 = 0.86$). Through years of field use, the PCI has continually been found to be an objective, repeatable scale (within ± 5 points with 95 percent confidence) of the collective judgment of experienced pavement engineers.

PREVENTIVE MAINTENANCE CONCEPTS

In a study conducted for USA-CERL by the Texas Transportation Institute/Texas A&M University (TTI/A&M), a comprehensive literature search was done to investigate existing strategies for the selection of M&R alternatives at various agencies *(5)*. Incorporated into the M&R algorithms used by the agencies is the selection of preventive maintenance treatments for pavements with little or no structural damage. Pavements with significant structural deficiencies must be rehabilitated with more appropriate M&R techniques. A summary of the approaches used by the California Department of Transportation, Texas State Department of Highways and Public Transportation (SDHPT), the San Francisco Bay Area, and the PAVER Pavement Management System is presented here.

In 1979, the California DOT implemented a PMS that featured M&R strategies based on the experience of agency engineers *(6, 7)*. The selection of a final repair technique for asphalt concrete pavements involves four basic steps. First, a survey is performed that determines the extent and severity of eight possible distress types. Each distress identified in a pavement section is entered into a decision tree to identify all possible solutions for the lane, as shown in Figure 2, for alligator, block,

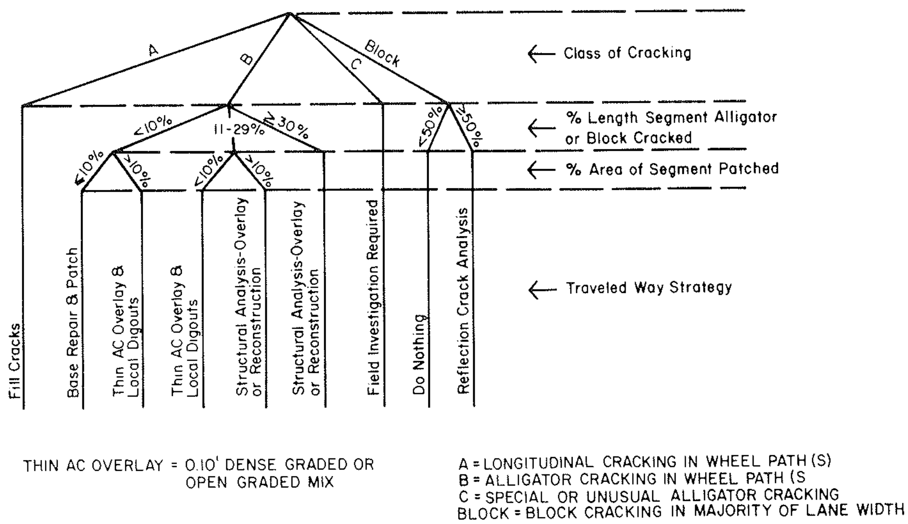

FIGURE 2 California DOT decision tree for flexible pavement alligator/block/longitudinal cracking.

or longitudinal cracking. Once alternatives have been identified, the second step is to compare each individual strategy and determine the one that will correct all the existing distresses in the lane. This strategy is referred to as the dominant strategy. The next step is to identify a compatible strategy that takes into consideration the dominant strategies for the shoulder and each pavement lane. Finally, all identified compatible strategies are listed in order of priority based on ride, distress rating, and average daily traffic (ADT).

The Texas SDHPT recently revised their PMS to include the selection of preventive maintenance requirements *(8, 9)*. Initially, their PMS was used as a network level tool that identified pavements in need of major M&R work and estimated budget requirements.

The decision on whether a pavement section is selected for rehabilitation is based on the pavement's condition rating. Texas uses a distress and serviceability rating consisting of seven surface distress types. A condition rating between 0 and 100 is determined with a score of 100 representing a pavement in perfect condition. Pavements identified as having a condition rating of less than 35 are flagged for rehabilitation consideration.

The Department found that a great deal of the work being proposed was for pavements with relatively high ratings (i.e., 55 to 75). Because these pavements were not identified as needing structural improvements, they were treated as candidates for preventive maintenance activities.

The preventive maintenance algorithm uses a decision trees procedure, such as that shown in Figure 3, which is based on the following criteria:

1. Pavement type (7 types of flexible pavements are defined),
2. Type and extent of pavement distress, and
3. Traffic level (4 levels are defined).

An appropriate maintenance strategy is identified for each pavement type/distress type/distress extent/traffic level combination. The maintenance strategies used in the Texas PMS are shown in Table 1.

Following the selection of possible alternatives, a dominant strategy is selected that ranks the selected strategies in order of their ability to repair multiple distresses. Once a procedure has been selected, the program then makes additional checks to identify the need for any necessary maintenance requirements.

The San Francisco Bay area PMS uses the PCI as an indicator of pavement condition *(4)*. In order to determine sections in need of M&R work, the most recent PCI is used with a PCI prediction technique to project the condition throughout a 5-year analysis period. These values are entered into selection criteria that specify the PCI ranges and deterioration rates for four M&R categories: major rehabilitation, overall rehabilitation, light rehabilitation, and a preventive maintenance program.

In this system, preventive maintenance is recommended for those sections with a PCI greater than 70 or a PCI that will not go below 70 in any of the first 3 years of the analysis program. Once identified as a preventive maintenance candidate, the present condition, projected condition, and any previously applied preventive maintenance treatments are all considered in the recommendation about which technique to apply. A series

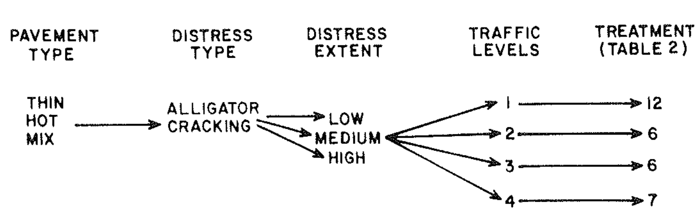

FIGURE 3 Example of one branch of Texas decision tree.

TABLE 1 MAINTENANCE STRATEGIES CONSIDERED IN THE TEXAS PAVEMENT EVALUATION SYSTEM

Maintenance Strategies	
Number	Name
0	Do Nothing
1	Seal Cracks
2	Partial Patch
3	Full Depth Patch
4	Fog Seal
5	Strip Seal
6	Seal Coat
7	Asphalt-Rubber Seal
8	Slurry Seal
9	Level-up
10	Thin Overlay
11	Rotomill
12	Spot Seal
13	Rotomill + Seal Coat
14	Rotomill + Thin Overlay

of treatments such as surface treatments, crack sealing, or skin patching is typically scheduled according to a predetermined time sequence.

A section will remain in the preventive maintenance program until either the PCI drops below 70 or a maximum allowable number of successive seal coats has been applied. Table 2 lists the preventive maintenance policy default values used in the PMS for crack seal intervals and maximum allowable number of successive seal coats. The user has the option of overriding the default values if, for example, a thin overlay is determined to be more appropriate than a seal coat on a high-volume road.

The existing PAVER PMS uses two tables, one each for flexible and rigid pavements, in the selection of appropriate maintenance strategies *(10)*. The guidelines for flexible pavements, as shown in Table 3, and those for PCC pavements, were intended to be applied to pavements with high PCI values. Using Table 3 as a starting point, the TTI/A&M study laid much of the groundwork in the development of a proposed preventive maintenance algorithm for the PAVER system. The selection of appropriate preventive maintenance treatments for candidate sections is based primarily on surface distress conditions and pavement rank. Pavement rank is used in the PAVER system as an indication of the functional classification of a pavement.

PREVENTIVE MAINTENANCE ALGORITHM DEVELOPMENT

As already mentioned, the objective of this paper is to outline the development of an algorithm for the recommendation of

TABLE 2 SAN FRANCISCO PREVENTIVE MAINTENANCE POLICY

Surface Type/ Branch Use	Crack Seal Interval	Number of Successive Seals Prior to Overlay or Removal	
		Chip Seals	Slurry Seals
AC ART	3 YRS	2	2
AC COL	4 YRS	2	3
AC RES/LOC	4 YRS	2	4
AC/AC ART	3 YRS	2	
AC/AC COL	4 YRS	2	
AC/AC RES/LOC	4 YRS	2	
AC/PCC ART	3 YRS	2	
AC/PCC COL	4 YRS	2	
AC/PCC RES/LOC	4 YRS	2	
ST COL	3 YRS		
ST RES/LOC	4 YRS		

Note: AC = asphalt surfaced, ART = arterial, AC/AC = asphalt surfaced overlaid with asphalt, COL = collector, AC/PCC = rigid pavement overlaid with asphalt, ST = surface treatment (armor coat) pavement, and RES/LOC = residential/local.

TABLE 3 PAVER PROCEDURE FOR IDENTIFYING M&R ALTERNATIVES FOR FLEXIBLE PAVEMENTS

Distress Type \ M&R Method	Do Nothing	Crack Seal	Partial Depth Patch	Full Depth Patch	Skin Patch	Pothole Filling	Apply Heat & Roll Sand	Apply Surface Seal Emulsion	Apply Rejuvenation	Apply Aggregate Seal Coat	Notes
1 Alligator Cracking			M,H	M,H				L	L		
2 Bleeding	L						L,M,H				
3 Block Cracking	L	L,M,H							L	L,M	
4 Bumps & Sags	L		M,H	M,H	M,H						
5 Corrugation	L		M,H	M,H							
6 Depression	L		M,H	M,H	M,H						
7 Edge Cracking	L	L,M	M,H	M,H							If predominant, apply shoulder seal, e.g., aggregate seal coat.
8 Joint Reflective Cracking	L	L,M,H	H								
9 Lane/ Shoulder Drop Off	L										If predominant level off shoulder and apply aggregate seal coat
10 Longitudinal Transverse Cracking	L	L,M,H	H					L	L	L,M	
11 Patching & Utility Cut	L	M	H*	H*							*Replace patch
12 Polished Aggregate	A									A	
13 Potholes			L	L,M,H		L,M,H					
14 Railroad Crossing	L				L,M,H						
15 Rutting	L		L,M,H	M,H	L,M,H						
16 Shoving	L		M,H								
17 Slippage Cracking	L	L	M,H								
18 Swell	L			M,H							
19 Weathering & Raveling	L		H					L,M	L	M,H	

Note: L = low severity; M = medium severity; H = high severity A = has only one severity level.

preventive maintenance strategies. The concepts described can be applied to both AC and PCC pavements, and can include any preventive maintenance treatments used by an agency. The initial algorithm is expandable so that environmental or geographical factors can be added at a later date.

Many factors are likely to influence the development of a preventive maintenance algorithm. Each agency must make decisions about appropriate strategies that conform to local policies and minimum pavement condition levels. Some of the factors that influenced the development of the algorithm used in this paper are presented here.

1. Agencies require a flexible algorithn. .at allows them to tailor the selection criteria according to local needs and policies.

2. It was found that most agencies will not typically apply seal coats and slurry seals to high-volume pavements. Often, crack seals and thin overlays are the only strategies recommended for this type of pavement. This assumption was adopted in the development of the algorithm.

3. Little information was available concern which preventive maintenance techniques work well under various environmental conditions.

4. Different climatic zones affect pavement deterioration rates in different ways. Factors such as amount of rainfall, type of subgrade, and number of freeze-thaw cycles influence the pavement deterioration rate if preventive maintenance is deferred. Average conditions were assumed in the development of the algorithm because insufficient information was available regarding the incorporation of climatic factors into the system. The proposed algorithm is flexible enough to allow for the addition of climatic factors as more data become available.

5. A trigger value should be set to indicate the lower boundary in the selection of candidate sections for preventive maintenance. A PCI default value of 70 was used as the trigger value in the development of this algorithm. Agencies will have the option of overriding the default value.

6. It was felt that small amounts of severe distress should not preclude a section from being selected for preventive maintenance. For that reason, patching was included as a repair procedure that could be recommended before the application of preventive maintenance activities.

7. All distress types identified in a section should be considered so that large amounts of severe distress, where restorative procedures may be more appropriate, are identified.

An effective preventive maintenance algorithm should consider certain fundamental steps. The flowchart shown in Figure 4 traces the logic used in the developed procedure. The concepts on which each step is based are outlined below.

Step 1: Define Parameters

The first step in the proposed algorithm is for the agency to define the parameters for the selection of candidate sections eligible for preventive maintenance and the strategies that should be included in the strategy selection tables. The agency would have the option of selecting preventive maintenance activities from the default strategy tables stored in the data base, or modifying these tables to fit local needs and policies. The default strategy tables may include preventive maintenance treatments such as crack sealing, patching, slurry seals, chip seals, and thin overlays. If strategy tables are developed for various pavement functional classifications, any restrictions on the use of certain activities on particular pavement ranks can be incorporated into the decision process.

Included in the development of strategy selection tables is the identification of upgrading strategies for any alternative excluded from consideration. The default upgrade strategy would permit slurry seals to be upgraded to chip seals, chip seals to be upgraded to thin overlays, and thin overlays to be upgraded to no preventive treatment (i.e., major rehabilitation) if the former treatment is excluded. Any modifications to the upgrading-strategy process should be made by the agency. This step ensures that only eligible treatments be considered throughout the remaining portion of the algorithm.

Another parameter that needs to be defined includes the minimum PCI above which a preventive maintenance strategy is recommended. Any sections with a PCI above the default value should be considered eligible for preventive maintenance. The recommended default value is a PCI of 70. Below this value, more corrective or structural types of M&R activities would normally be required.

Finally, unit costs for each of the possible preventive maintenance treatments should be entered. Generally, costs are entered in terms of dollars per square foot, with the exception of some activities (e.g., crack sealing) that would be more appropriate in units of dollars per linear foot.

Step 2: Check Eligibility

Once the parameters have been defined, the data base is searched to determine which sections fall within the established parameters. In addition to checking the defined parameters, the suitability of applying preventive maintenance treatments to each section should be examined by asking questions that may alert the agency to any unusual conditions. Typical questions could include the following:

1. Does the latest PCI of the section fall outside the specified (or default) PCI range?
2. Is the required AC overlay thickness needed for the section greater than 2.5 in.?
3. Does the pavement have a high deterioration rate?

If the answer to any of these questions is yes, the section should no longer be considered for preventive maintenance and project level investigations should be performed.

Step 3: Generate Density/Severity Classifications

After the candidate sections have been identified, density/severity classifications should be determined. Unlike other preventive maintenance algorithms, which base density ranges on the subjective judgment of a few engineers, this procedure, developed at USA-CERL, relates distress density ranges directly to PCI deduct values. Three ranges of PCI deduct points corresponding to an acceptable amount of distress should be

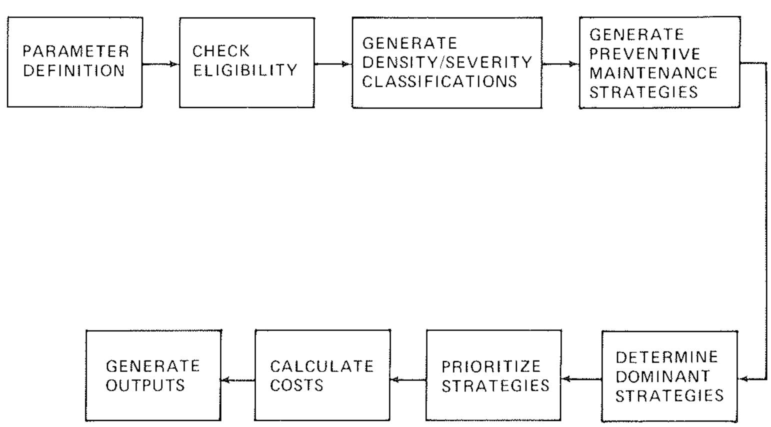

FIGURE 4 Preventive maintenance algorithm flowchart.

established for each of the three density levels: low, medium, and high. To accomplish this, the densities corresponding to each of the deduct ranges must be determined for all distress type/severity level combinations. The advantage to this approach is that the amount of deterioration that each distress type/severity level combination causes on the pavement is derived from an objective rating scale that was used to determine PCI deduct values.

Because deduct values serve as a type of weighting factor that indicates the size of the effect that the particular distress type/severity level has on pavement condition, they can be used as a quantifiable indication of the amount of damage allowed within each of the three density ranges for each defined distress type and severity level. Through extensive use of the PCI, the reliability of the deduct curves to represent the subjective rating of experienced pavement engineers has been accepted.

To demonstrate the use of this concept in a preventive maintenance algorithm, polynomial curve-fitting techniques developed at USA-CERL were used to derive equations for the PAVER PCI deduct curves. A total of 104 curves were fitted for each of the defined distress type/severity level combinations for asphalt concrete (AC) and PCC pavements. As an example, Figure 5 shows a fourth-order fit for low-severity alligator cracking. If fourth-order curves were not found to be acceptable, fifth- and sixth-order curves were generated to procure the best possible fit. After the best fit curves were found, equations were written for each curve. As can be seen in the figure, this technique resulted in excellent curve fits.

A computer program that back-calculates distress densities for any deduct value was developed using the AC deduct curve equations. Densities were determined for all AC distress type/severity level combinations at various deduct values. An example of a portion of the output from the program is shown in Table 4.

Initial density ranges based on the deduct value concepts presented above were developed based on input from several pavement engineers. A low-density range was defined for distress type/severity level combinations corresponding to a deduct value less than or equal to 10 points. Distress type/severity level combinations with a range of deduct values between 10 and 20 points made up the medium-density classification range, and distress combinations that resulted in deduct values greater than or equal to 20 points were assigned to the high-density classification range. The resulting low-, medium-, and high-density ranges are shown in Tables 5, 6, and 7 for low-, medium-, and high-severity asphalt concrete distress types, respectively. Future research should include obtaining the opinions of additional pavement engineers for modifications to the initial density ranges. In areas where PCI deterioration rates vary from the average rate because of climatic or other conditions, the density range tables can easily be designed to suit local conditions.

To use the tables, each distress type/severity level identified in the condition survey is located in the appropriate severity level table. From these tables, corresponding density classifications

TABLE 4 PAVER DISTRESS DENSITY/DEDUCT VALUES

Distress Type	Severity Level	Density (%)
Alligator Cracking	Low	0.28913
Bleeding	Low	16.91303
Bleeding	Medium	2.03118
Bleeding	High	0.94241
Block Cracking	Low	5.49324
Block Cracking	Medium	1.96720
Block Cracking	High	0.86778
Bumps and Sags	Low	0.69500
Corrugation	Low	2.85177
Corrugation	Medium	0.10000
Depression	Low	1.85847
Edge Cracking	Low	2.80141
Edge Cracking	Medium	0.27842
Jt Reflection Cracking	Low	2.28622
Jt Reflection Cracking	Medium	0.73506
Jt Reflection Cracking	High	0.20625

Note: Distress densities correspond to a deduct value of 5.00.

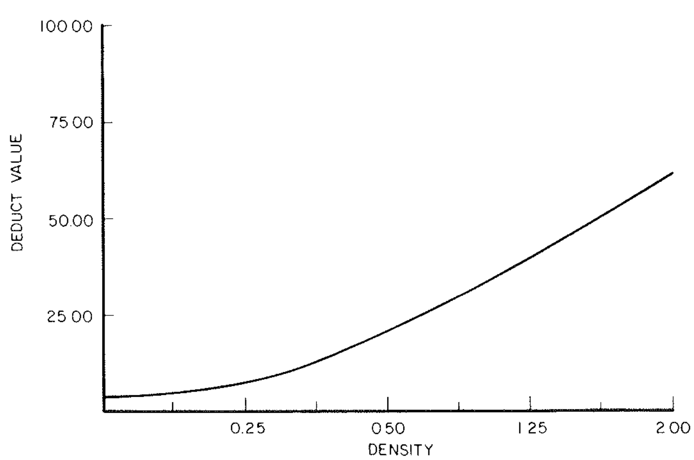

FIGURE 5 Fourth-order fit curve for low-severity alligator cracking.

TABLE 5 DENSITY RANGES FOR LOW-SEVERITY DISTRESSES

Distress Type	Low Density	Medium Density	High Density
Alligator Cracking	< 1	1 - 3	> 3
Bleeding	< 38	>38	No Data
Block Cracking	< 14	14 - 48	> 48
Bumps and Sags	< 2	2 - 4	> 4
Corrugation	< 7	7 - 20	> 20
Depression	< 5	5 - 11	> 11
Edge Cracking	< 9	>9	No Data
Jt. Reflection Cr	< 6	6 - 18	> 18
Lane/Shoulder Dropoff	< 8	>8	No Data
L/T Cracking	< 5	5 - 14	> 14
Patch/Utility Cut	< 5	5 - 15	> 15
Polished Aggregate	< 40	>40	No Data
Potholes	< 0.05	0.05 - 0.10	> 0.10
RR Crossing	< 8	8 - 50	> 50
Rutting	< 1	1 - 5	> 5
Shoving	< 3	3 - 10	> 10
Slippage Cracking	< 2	2 - 6	> 6
Swell	< 7	7 - 30	> 30
Weathering & Raveling	< 32	>32	No Data

Note: Low severity distress (% distress density). Jt Reflection Cr = joint reflection cracking; L/T Cracking = longitudinal/transverse cracking; RR = railroad.

TABLE 6 DENSITY RANGES FOR MEDIUM-SEVERITY DISTRESSES

Distress Type	Low Density	Medium Density	High Density
Alligator Cracking	< 0.20	0.20 - 1.0	> 1
Bleeding	< 6	6 - 24	> 24
Block Cracking	< 5	5 - 16	> 16
Bumps and Sags	< 0.20	0.20 - 0.70	> 0.70
Corrugation	< 0.50	0.50 - 2.0	> 2
Depression	< 2	2 - 6	> 6
Edge Cracking	< 2	2 - 6	> 6
Jt Reflection Cr	< 2	2 - 4	> 4
Lane/Shoulder Dropoff	< 5	5 - 10	> 10
L/T Cracking	< 1	1 - 4	> 4
Patch/Utility Cut	< 1	1 - 4	> 4
Polished Aggregate	< 40	> 40	No Data
Potholes	< 0.02	0.02 - 0.04	> 0.04
RR Crossing	< 2	2 - 4	> 4
Rutting	< 0.3	0.30 - 1.0	> 1
Shoving	< 1	1 - 3	> 3
Slippage Cracking	< 1	1 - 2	> 2
Swell	No Data	< 3	> 3
Weathering & Raveling	< 2	2 - 12	> 12

Note: Medium severity distresses (% distress density).

are assigned based on the total quantity of the particular distress type/severity level combination tabulated for the section. Each density classification is then located in a density/severity classification code table such as the one shown in Table 8. This table assigns a number from 1 to 9, which corresponds to a possible preventive maintenance strategy, as explained in the next step.

For example, if a distress survey on one section indicates that there is 5 percent low-severity alligator cracking and 3 percent medium-severity edge cracking present, density classifications of high and medium would be assigned from Tables 5 and 6, respectively. Entering Table 8 for the alligator cracking with a high-density classification and low-severity level, gives a density/severity classification code of 3. Repeating the procedure for the medium-severity edge cracking with a medium-density classification gives a density/severity classification code of 5. These values will be used in the next step to determine appropriate preventive maintenance strategies.

Step 4: Generate Preventive Maintenance Strategies

The distress density/severity classification code identified in the previous step for each distress type is entered into an appropriate strategy-selection table. An excerpt from a typical table for primary and secondary roads is shown in Table 9. These tables should be developed for various pavement functional classifications and should include legitimate preventive maintenance strategies. Commonly used strategies for AC pavements typically include crack sealing, chip seals, slurry seals, patching, thin overlays, do nothing, and no appropriate preventive maintenance strategy. The default table is modified according to the parameters established in Step 1 so that a customized recommendation can be made. The alternatives shown in Table 9 were selected by combining the existing procedures for identifying M&R alternatives in the PAVER system (Table 3) with the experienced judgment of pavement engineers.

As a result of this step, appropriate preventive maintenance strategies are specified for each distress type/severity level identified in the latest condition survey. If the selected treatment was excluded from consideration in Step 1, it should now be upgraded to a treatment defined within the established parameters. If, for example, a chip seal is identified as the recommended strategy for a particular distress type on a high-volume pavement but local policy prevents the use of this type of

TABLE 7 DENSITY RANGES FOR HIGH-SEVERITY DISTRESSES

Distress Type	Low Density	Medium Density	High Density
Alligator Cracking	No Data	< 0.5	> 0.5
Bleeding	< 3	3 - 8	> 8
Block Cracking	< 2	2 - 5	> 5
Bumps and Sags	No Data	< 0.1	> 0.1
Corrugation	< 0.1	0.1 - 0.2	> 0.2
Depression	No Data	< 2	> 2
Edge Cracking	< 0.6	0.6 - 2.0	> 2
Jt Reflection Cr	< 0.5	0.5 - 2.0	> 2
Lane/Shoulder Dropoff	< 2	2 - 5	> 5
L/T Cracking	< 0.4	0.4 - 1.0	> 1
Patch/Utility Cut	< 0.3	0.3 - 1.0	> 1
Polished Aggregate	< 40	> 40	No Data
Potholes	No Data	No Data	> 0.01
RR Crossing	No Data	< 1	> 1
Rutting	< 0.2	0.2 - 0.5	> 0.5
Shoving	< 0.2	0.2 - 1.0	> 1.0
Slippage Cracking	< 0.4	0.4 - 1.0	> 1.0
Swell	No Data	No Data	> 1
Weathering & Raveling	< 0.2	0.2 - 2.0	> 2

Note: High severity distresses (% distress density).

TABLE 8 DENSITY/SEVERITY CLASSIFICATION CODES

		DISTRESS DENSITY		
		L	M	H
DISTRESS SEVERITY	L	1	2	3
	M	4	5	6
	H	7	8	9

TABLE 9 EXCERPT FROM STRATEGY SELECTION TABLE: PRIMARY AND SECONDARY ROADS

DISTRESS TYPE: ALLIGATOR CRACKING	
Classification Code	Preventive Maintenance Strategy
1	DO NOTHING
4,7	PATCHING
2,3,5,6	THIN OVERLAY
8,9	NONE PREVENTATIVE

treatment, the recommended strategy should be upgraded to a thin overlay (or to the appropriate treatment, as identified in Step 1).

Using the same example as in the previous step, and applying the recommendations found in Table 9, the suggested strategy for the alligator cracking would be Thin Overlay. Similar strategy tables would exist for all other distress types.

Step 5: Determine Dominant Strategies

After all possible preventive maintenance alternatives have been identified for a section, one dominant strategy needs to be selected. This step should include the formation of a flow chart, which includes all possible combinations that could be selected for a section identified as a preventive maintenance candidate. The flowchart must distinguish which alternatives override others and which combinations give an indication that preventive maintenance may not be appropriate, and a project level investigation should be performed. An example of a flowchart is shown in Figure 6.

The dominant strategy selected should also include any localized corrective treatments such as crack sealing or patching, which need to be applied before the recommended alternative is applied. Quantities of work should also be determined for the calculation of costs as described in a later step.

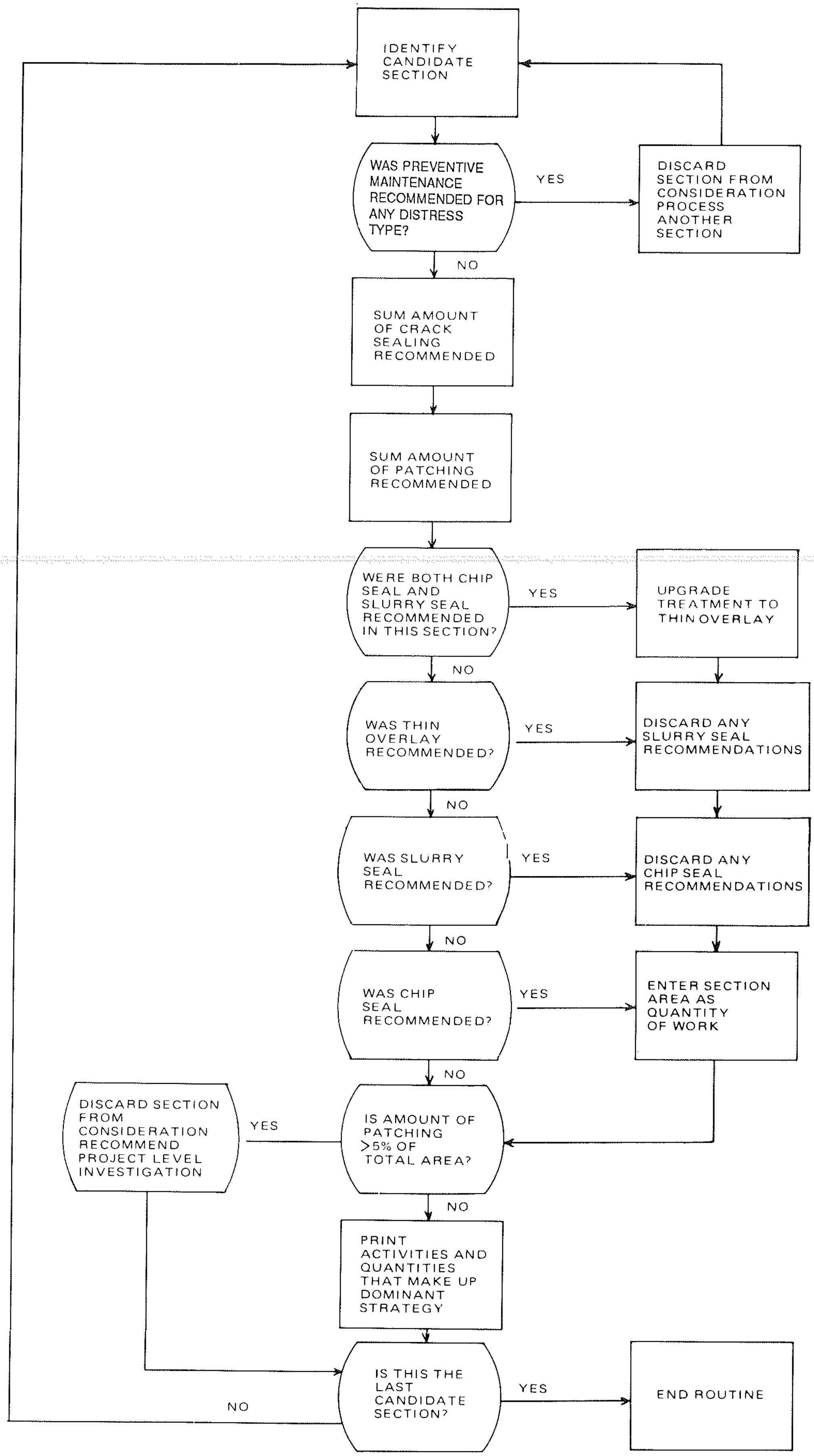

FIGURE 6 Dominant strategy selection flowchart.

PCI \ PAVEMENT RANK	PRIMARY	SECONDARY	TERTIARY
90 - 100	4	7	9
80 - 89	2	5	8
70 - 79	1	3	6

FIGURE 7 Preventive maintenance prioritization scheme.

Step 6: Prioritize Strategies

Once the final decision has been made about the appropriate dominant strategy, the agency must devise a ranking scheme so that the most crucial projects are funded first. Typically, parameters used in the decision process include some indicator of condition and functional classification of the pavement. A sample prioritization scheme is shown in Figure 7. This scheme places the emphasis on applying preventive maintenance treatments to eligible sections with the highest functional classification (primary) and worst condition ($70 < PCI < 80$). Any remaining money is then allocated to candidate sections on primary roads with PCIs between 80 and 90. Additional projects continue to be scheduled in order of priority until all preventive maintenance funding is depleted.

Step 7: Calculate Costs

Based on the costs' input in Step 1, an optimal preventive maintenance budget should be prepared that determines the costs associated with applying the selected treatment to each candidate section. These costs will be used in the next step to determine actual preventive maintenance projects.

Step 8: Generate Outputs

The results of the previous seven steps are summarized and presented in the form of a preventive maintenance report outlining work to be performed and budget estimates. By combining the prioritized work list obtained in Step 6 and the cost figures obtained in Step 7 with the actual amount of dollars available, a list of actual preventive maintenance projects will be obtained.

SUMMARY

A Pavement Management System is an important tool for pavement engineers in maintaining pavements in the best possible condition for the lowest cost. By performing pavement repairs while the pavements are still in good condition, costs can be reduced by a factor of 4 to 5. The application of preventive maintenance treatments can play an important role in prolonging pavement service life by deterring pavement deterioration. This paper presented the development of a preventive maintenance algorithm for use in Pavement Management Systems and introduced a new concept in determining distress density limits for the recommendation of preventive maintenance treatments.

The preventive maintenance algorithm outlined in this paper consists of several fundamental steps. First, parameters are defined. This includes identifying which treatments are to be considered and the unit costs associated with these activities, in addition to the minimum pavement condition above which a preventive maintenance strategy is recommended. Eligible sections are identified, and distress density classifications are determined.

Unlike other preventive maintenance algorithms, which base density ranges on subjective judgments, a procedure is described in this paper that was developed at USA-CERL that relates distress density directly to PCI deduct values. The advantage to this approach is that the amount of deterioration that each distress type/severity level combination causes on the pavement is derived from an objective rating scale that was used to determine PCI deduct values.

After candidate sections have been identified and density classifications have been assigned, preventive maintenance treatments are recommended. A dominant strategy is selected from all possible treatments for a section, and costs are calculated. Based on the agency prioritization scheme and the budget available, actual preventive maintenance projects can be identified.

The preventive maintenance concepts outlined in this paper are applicable to both AC and PCC pavements. They are designed to be flexible enough to allow for local policies and conditions. In addition, they can be easily expanded to include environmental or geographical factors and additional preventive maintenance treatments at a later date.

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Pavement Performance Prediction Model Using the Markov Process

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A good pavement-management system requires an accurate and efficient pavement performance and prediction model. A pavement performance and prediction model based on the Pavement Condition Index and the age of the pavement has been developed. The Pavement Condition Index ranging from 0 to 100 has been divided into ten equal condition states. A combination of homogeneous and nonhomogeneous Markov chains has been used in the development of the model. The life span of the pavement is divided into zones, with each zone representing a period of 6 years. The transition matrix of each zone is determined using nonlinear programming. If the state of any given pavement section is known, its future condition can be predicted efficiently from the corresponding transition matrices. The model presented in this paper will play an integral part in the decision-making procedure for determining optimal maintenance and repair strategies. A comparison between the Markov model and the constrained least-squares model is presented.

The performance of existing pavements and the prediction of their future conditions is a matter of great concern to pavement engineers. In recent years there has been a rapid growth in the technology of pavement evaluation and rehabilitation. A corresponding growth has occurred in the development of Pavement Management Systems (PMS), which are based on the performance of the existing pavements. These developments necessitate more reliable pavement performance and prediction models. Knowledge about the future condition of the pavement is required for inspection scheduling, life cycle costing, benefit analysis, and budget optimization. A pavement performance and prediction model based on Pavement Condition Index (PCI) and age has been developed. The PCI ranging from 0 to 100 has been divided into ten equal states and a combination of homogeneous and nonhomogeneous Markov chains has been used. This prediction model has the potential to be an integral part in the decision-making procedure for determining optimal maintenance and repair strategy.

CURRENT PREDICTION MODELS

The prediction models currently in use vary in complexity from simple straight-line extrapolation *(1)* to probability-based models *(2)*. Straight-line extrapolation is used to predict the condition of a pavement section. When sufficient data are available, it is found that the shape of the deterioration curve is generally curvilinear rather than the straight line that results from straight-line extrapolation.

In other attempts the regression techniques have been used to model the pavement condition deterioration over time *(3–5)*. These regression techniques are valid only if the predictive variables can be found that are related to pavement condition deterioration. The regression techniques are applicable only to specific climatic conditions, materials, construction techniques, and others.

Recently, researchers at U.S. Army Construction Engineering Research Laboratory (USA-CERL) have investigated two other mathematical techniques for curve fitting *(6)*: the constrained least-squares and the *B*-spline. The constrained least-squares model fits a polynomial curve to the data that minimizes the squared distance between the predicted and the actual data points. At the same time the technique applies a constraint that ensures a monotonically decreasing slope of the predicted condition versus age curve.

The *B*-spline method is based on the original mechanical splines used in drafting and it assumes that the curve takes on a shape that minimizes its potential energy. A *B*-spline of degree k is a continuous function having its first k-1 derivatives continuous. Because of the complex nature of selecting the number and the position of interior knots and the possibility of the occurrence of a positive slope in the function, the *B*-spline technique is not deemed suitable as a pavement condition prediction-modeling technique. The constrained least-squares curves, unlike *B*-spline curves, never exhibit a positive slope; i.e., the PCI values are not allowed to increase with age.

From this initial evaluation, the constrained least-squares estimation approach was selected to model the relationship between PCI and age. Figure 1 shows the curve-fitting results with constrained least-squares estimation and *B*-spline approximation. The *B*-spline curve shows a positive slope, whereas the constrained least-squares curve more accurately predicts the normal pavement deterioration behavior.

The probability-based Markov model was first developed for the Arizona PMS *(2)* to describe pavement condition changes. Intuitively, the behavior of pavements is not deterministic but is probabilistic. Consequently, the selection of an appropriate repair strategy is also an uncertain procedure. Because of the probabilistic nature of pavements, it was decided to develop a probability-based prediction model, as outlined in the next section.

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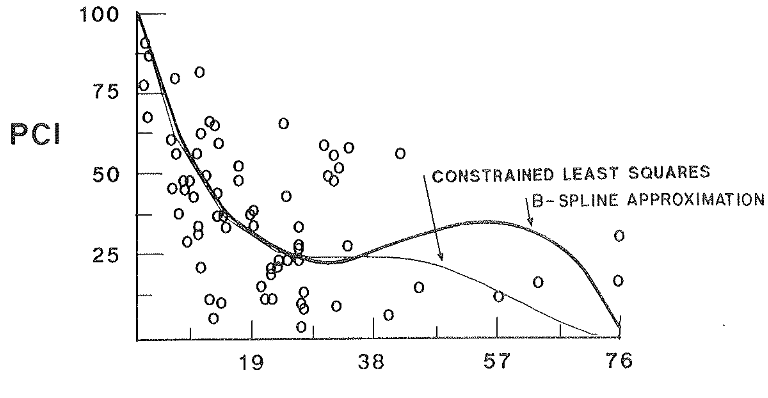

FIGURE 1 Constrained least squares versus *B*-spline approximation.

ADVANTAGES OF PROBABILITY-BASED PREDICTION MODELS

Kulkarni *(7)* outlined several advantages in using a Markov probability decision process in pavement management. These include the following:

1. Future decisions on preservation actions are not fixed but depend on how the pavements actually perform.
2. Actions to be taken now can be identified; also, likely actions to be taken in the next few years can be identified with a high degree of probability.
3. It is possible to compare the expected proportions in given condition states with the actual proportions observed in the field, and in this way possible defects in construction, materials, quality control, and so on, can be identified.
4. A dynamic decision model has the potential for significant cost savings by selecting less conservative rehabilitation actions that will still satisfy the prescribed performance standards.

Straight-line extrapolation techniques are deterministic and do not attempt to explain the variability among the data points; they merely fit a best line to the data. Regression techniques are powerful tools, but in many cases the models are chosen for the best fit without regard to the suitability or intrinsic relevance of the variables selected. Polynomials of different degrees and mathematical functions can be manipulated to fit the data; but when these functions are projected beyond the bounds of the data the results can be totally misleading.

It is known that the rate of deterioration is uncertain. Therefore, the predictive model should portray this rate of deterioration as uncertain, rather than using the erroneous assumption of deterministic behavior. The Markov process imposes a rational structure on the deterioration model. This form of predictive model has the further advantage of ensuring that projections beyond the limits of the data will continue to have the classic pattern of worsening condition with age, something that the regression models cannot guarantee.

Another advantage of probability-based models is the ease with which they can be integrated into optimization processes. The Markov process is a natural tool to use in alliance with dynamic programming to produce optimal solutions. It is believed that the application of the Markov process in conjunction with dynamic programming will produce optimal maintenance and rehabilitation (M&R) strategies for selected pavement sections quickly and efficiently.

CONCEPTS UTILIZED

Pavement Condition Index

The procedure used to measure the performance of the existing pavements is the PCI, developed by USA-CERL *(8)*. PCI is a composite index of the pavement's structural integrity and operating condition. The PCI of a pavement is determined from a detailed survey that measures distress type, severity, and quantity to produce a numerical index ranging from 0 to 100, with 100 being excellent.

Pavement Family Classification

The PAVER data base was used for the development of the prediction model *(1)*. Every pavement section stored in the PAVER data base is identified by the location, the pavement type, the pavement use, and the pavement rank or functional classification. The family concept of grouping similar pavement sections, as shown in Figure 2, is used to account for the variety of factors affecting the pavement performance. A pavement family is defined as the group of pavement sections with the same pavement type, the pavement use, and the pavement rank.

The desired (ideal) form of data for determining the rate of deterioration is shown in Figure 3. In this form, all the pavement sections are put into use at the same time. Unfortunately, there are very few pavement sections for which complete pavement condition history data are available. Therefore, the approach taken was to survey all pavement sections of various ages of a family at the same time, as shown in Figure 4*a*.

An assumption was made that these sections represent the condition of a pavement section at the various ages as shown in Figure 4*b*. Of course, when sufficient information is available over time for each pavement section, there will be more confidence in the predicted curves.

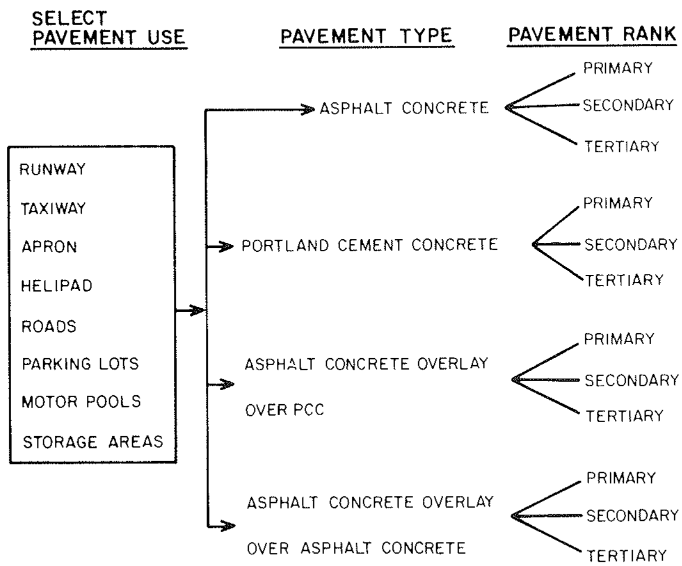

FIGURE 2 Family definition from PAVER data base.

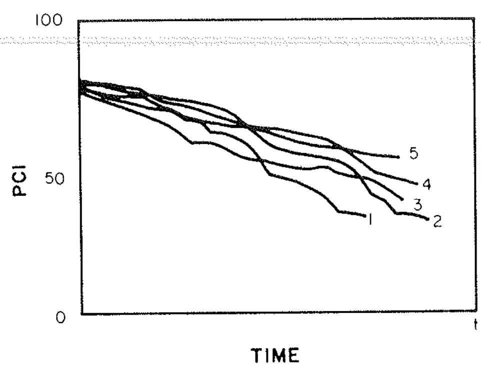

FIGURE 3 Desired (ideal) form of data.

Data Errors Screening (Filtering)

There are errors in the family data retrieved from the PAVER data base. These errors might have originated during data collection, coding, or entering the data into the data base. A computerized filtering program, developed at USA-CERL, is used to identify obviously erroneous data such as duplicate records, same age and different PCI for a given section, PCI greater than 100, and unrealistic PCI values. The user has the option of examining the erroneous data file and making adjustments to the filter boundaries.

Outliers Identification

An outlier analysis program developed at USA-CERL *(6)* is used to remove the extreme observations. The extreme data points have substantial impact on modeling the family behavior. The outlier program fits a constrained least-squares curve to the filtered family data file and sets confidence limits for the residuals of given observations; e.g., 95 percent, to be established by the user for removal of extreme cases.

MODELING APPROACH

The model development process is made up of the following steps:

- Data retrieval by pavement family
- Data errors screening (filtering)
- Outliers identification
- Development of Markov model

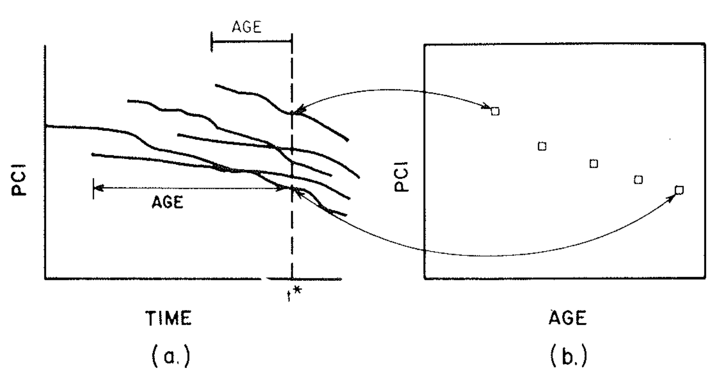

FIGURE 4 Relationship between PCI and age of all pavement sections of a family surveyed at the same time.

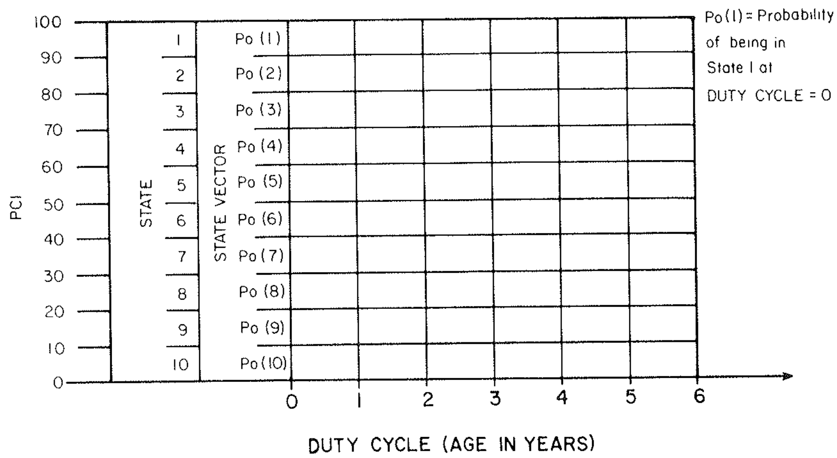

FIGURE 5 Schematic representation of state, state vector, and duty cycle.

DEVELOPMENT OF MARKOV MODEL

A pavement begins its life in nearly perfect condition and is then subjected to a sequence of duty cycles that cause the pavement condition to deteriorate. In this study the state of a pavement is defined in terms of PCI rating. The PCI ranging from 0 to 100 has been divided into ten equal states, each state being 10 PCI points wide. A duty cycle for a pavement is defined as one year's duration of weather and traffic. A state vector indicates the probability of a pavement section being in each of the ten states in any given year. Figure 5 is the schematic representation of state, state vector, and duty cycle.

After filtering and outlier analysis, all the surveyed pavement sections of a family are categorized into one of the ten states at any age. It is assumed that all the pavement sections are in State 1 (PCI of 90 to 100) at an age of 0 yr. Thus, the state vector in duty cycle 0 (age = 0) is given by (1, 0, 0, 0, 0, 0, 0, 0, 0, 0), as it is known (with probability of 1.0) that the pavement sections must lie in State 1 at an age of 0 yr.

To model the way in which the pavement deteriorates with time, it is necessary to identify the Markov probability transition matrix. In the present case, the assumption is made that the pavement condition will not drop by more than one state (10 PCI points) in a single year. Thus, the pavement will either stay in its current state or transit to the next lower state in one year. Consequently, the probability transition matrix has the form:

$$P = \begin{matrix} p(1) & q(1) & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & p(2) & q(2) & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot & \cdot \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p(9) & q(9) \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{matrix}$$

where $p(j)$ is the probability of a road staying in state j during one duty cycle, and $q(j) = 1 - p(j)$ is the probability of a road transiting down to the next state $(j + 1)$ during one duty cycle.

The entry of 1 in the last row of the transition matrix corresponding to State 10 (PCI of 0 to 10) indicates a holding or trapping state. The pavement condition cannot transit from this state unless repair action is performed.

The state vector for any duty cycle, t, is obtained by multiplying the initial state vector $\bar{p}(0)$ by the transition matrix P raised to the power of t. Thus,

$$\begin{aligned} \bar{p}(1) &= \bar{p}(0) \times P \\ \bar{p}(2) &= \bar{p}(1) \times P = \bar{p}(0) \times P^2 \\ &\;\vdots \\ \bar{p}(t) &= \bar{p}(t-1) \times P = \bar{p}(0) \times P^t \end{aligned}$$

With this procedure, if the transition matrix probabilities can be estimated, the future state of the road at any duty cycle, t, can be predicted.

To estimate the transition matrix probabilities, the Fletcher-Powell algorithm *(9)*, a nonlinear programming approach, is used. The objective of the search is to determine values of the nine parameters, $p(1)$ through $p(9)$, that would minimize the absolute distance between the actual PCI versus age data points, and the expected (predicted) pavement condition for the corresponding age generated by the Markov chain using these nine parameters.

The objective function has the following form:

$$\text{MIN} = \sum_{t=1}^{N} \sum_{j=1}^{M(t)} \left| Y(t,j) - E[X(t,p)] \right|$$

where

N = total number of duty cycles (age) for which PCI versus age data are available within each family;

$M(t)$ = total number of data points recorded at a duty cycle (age) t;

$Y(t, j)$ = PCI rating for each sample taken at a duty cycle (age) t; and

$E[X(t, p)]$ = expected value in PCI at a duty cycle (age) t, as predicted by the current Markov values.

ORIGINAL MARKOV MODEL

The original Markov model for pavement deterioration was developed under a contract to USA-CERL by Keane and Keane *(10)*. In the initial investigation, the objective was to make the model as simple as possible. Therefore, the number of states

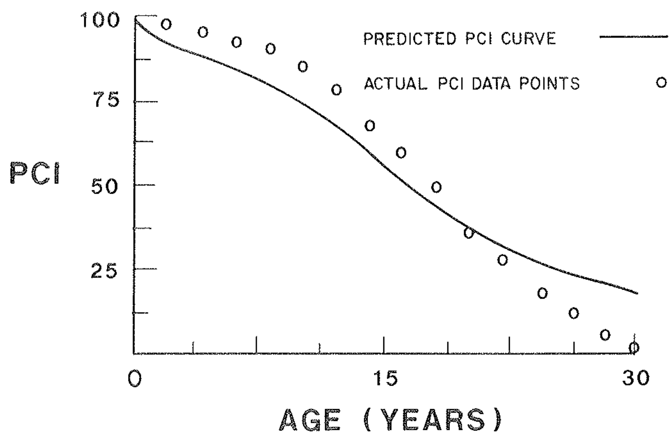

FIGURE 6 Pavement condition prediction curve using original Markov model.

was chosen to be eight and only one state transition was allowed during a duty cycle. Also, the Markov chain was assumed to be homogeneous or stationary; that is, the duty cycle was taken to be constant over time.

VALIDITY OF THE ORIGINAL MARKOV MODEL

The validity of the original Markov model was checked using a test data file representing actual PCI data points over a 30-year life of the pavement. The final results from the original model are shown in Figure 6. A significant difference between the predicted PCI values and the actual values is noticeable in this figure. It was concluded from this experiment that the original model was not capable of accurately matching the actual deterioration curve of the pavement.

This discrepancy in the original model is attributed to the incorrect assumption of a constant duty cycle over the life of the pavement. Traffic loads generally increase over time, which means that the duty cycle will have become successively more destructive each year. It should be noted here that the Markov prediction curves are developed for the family data files only. Therefore, the pavement type, traffic, and climate are already taken into account in family definition. The increase in the traffic loads, as already mentioned, is the gradual increase within a given traffic category; i.e., primary, secondary, or tertiary. In the development of the original model it was assumed that the pavement condition would not decrease more than 10 PCI points in a single year, and only one state transition was allowed. The additional assumption of eight states' division contradicts the assumption that the pavement condition would not decrease more than 10 PCI points in a single year because the last two states were made up of 20 PCI points each.

NEW MARKOV MODEL

In the new model a more refined definition of the states has been used. The number of states has been increased from eight to ten, each state covering 10 points on the PCI scale. To allow for changes in traffic loads and maintenance policies over the pavement life, different duty cycles have been introduced in the new model. An ideal approach for the model is to have a different duty cycle for each year. Because of the limited availability of yearly PCI data, this was not feasible. To achieve the result of having different duty cycles, a zoning scheme has been developed in which the life of the pavement has been divided into zones, each zone representing a period of 6 years. It is assumed that each zone has a constant rate of deterioration; hence a constant duty cycle has been assumed within each zone. The rate of deterioration is assumed to vary from one zone to another; therefore, different duty cycles have been assigned to different zones. The 6-year period of a zone is a realistic assumption as a PCI survey is performed every 3 years, on the average. This sequence provides two section-level PCI condition survey points within each zone.

As the duty cycle within a zone is assumed to be constant, a homogeneous Markov chain and a separate transition matrix have been developed for each zone. The duty cycle varies from one zone to another. Therefore, a nonhomogeneous Markov chain has been used for transition from one zone to another. Zone 1 is always assumed to start in State 1 with state vector (1, 0, 0, 0, 0, 0, 0, 0, 0, 0). Zone 2 takes the last state vector of Zone 1 as its starting state vector. This process continues for all the zones over the life of the pavement. This procedure ensures a continuous curve over the pavement life.

VERIFICATION OF NEW MARKOV MODEL

The validity of the new Markov model was verified using the same test data file that was used for the original model. The final results from the new model are shown in Figure 7. It is clear from this figure that the new Markov model predicts PCI values much closer to the actual PCI values than the original Markov model. The Markov model presented in this paper was tested using a large number of different data files. Examples of the results for two of these files are shown in Figures 8 and 9.

The Markov modeling procedure first sorts the actual PCI values by age and then groups them into zones. The state vector and transition matrix are determined separately for each zone. The expected PCI values for each year are determined from the state vector and the transition matrix of the given zone. The Markov model is very sensitive to initial starting values for the transition matrix probabilities. The rate of deterioration varies

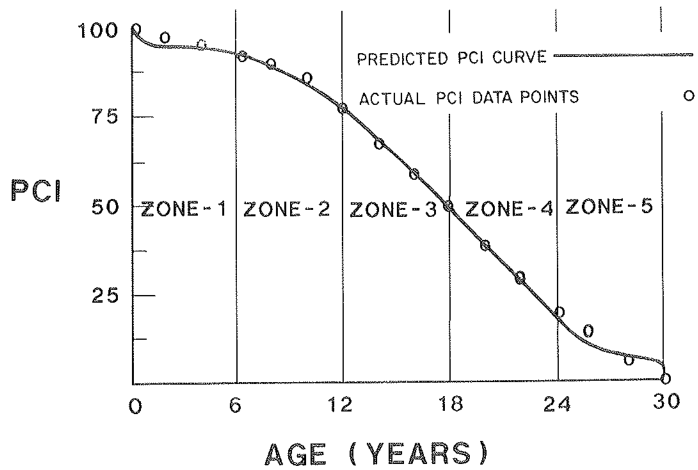

FIGURE 7 Pavement condition prediction curve using new Markov model.

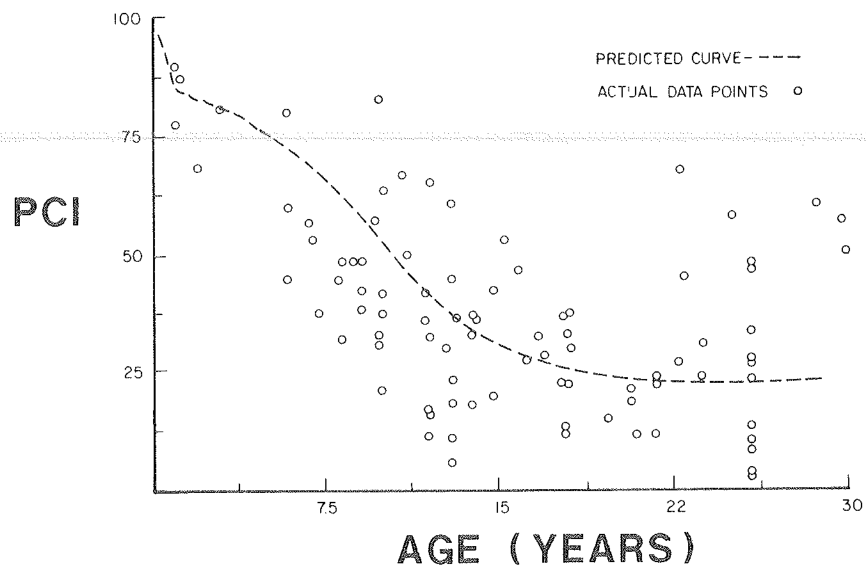

FIGURE 8 Example of pavement condition prediction curve using new Markov model.

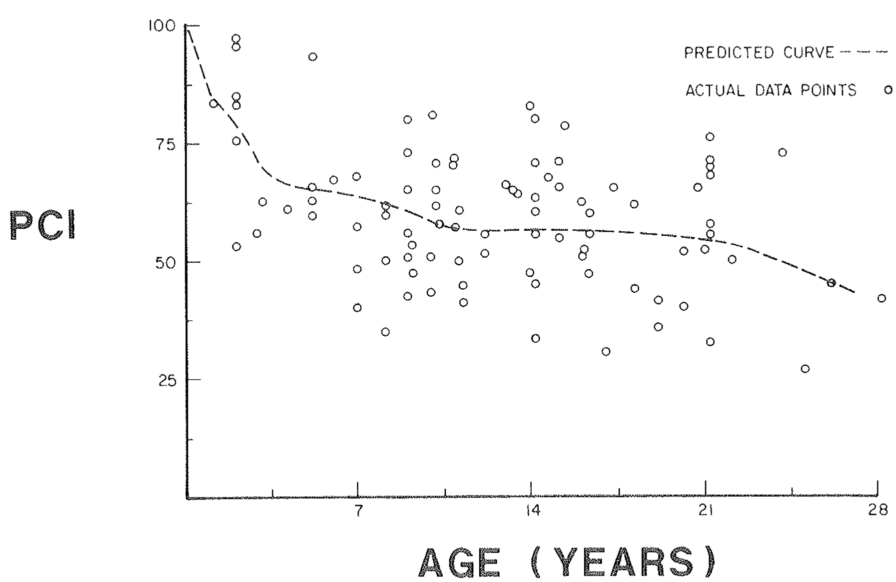

FIGURE 9 Example of pavement condition prediction curve using new Markov model.

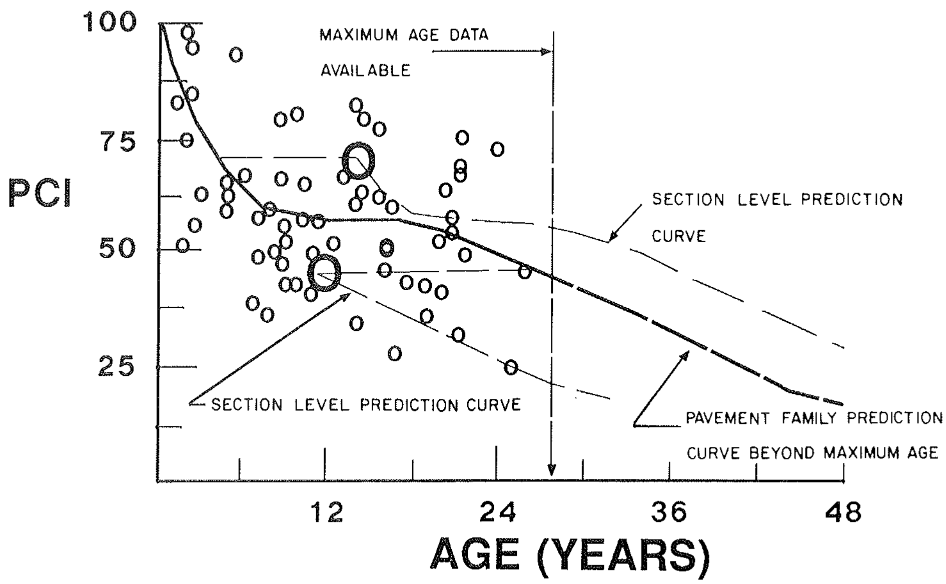

FIGURE 10 Pavement condition prediction curves using new Markov model.

from zone to zone, therefore different initial starting values for the transition matrix probabilities are used for different zones. The computer time and the number of iterations are reduced significantly by using different input starting values for different zones.

FUTURE PREDICTION USING NEW MARKOV MODEL

Information about the condition of the pavement in the future is needed for life-cycle cost analysis at the project level and for developing optimal M&R strategies at the network level. The capability of future prediction is required for pavement family curves and for each individual section. The Markov model is the only technique that is capable of predicting the condition of the pavement beyond the last available data point by using the transition matrix of the last zone. The section level prediction is carried out by first determining the present state of the section and then projecting the future condition by using the transition matrices of the respective zones. The pavement family prediction curve beyond the maximum age and the pavement section level prediction curves are shown in Figure 10.

COMPARISON OF NEW MARKOV MODEL WITH CONSTRAINED LEAST-SQUARES MODEL

Comparison of the curve-fitting results for the new Markov model and the constrained least-squares model is shown in Figure 11. The curves from the two different techniques show almost the identical trends of the pavement performance. Comparison of the extrapolation results from the two techniques is shown in Figure 12. The extrapolation curves from the two different techniques are significantly different. The extrapolation curve from the new Markov model is the most likely to represent the future condition of the pavement. The new Markov model is preferred to the constrained least-squares model because it is best for extrapolation. Also, the new

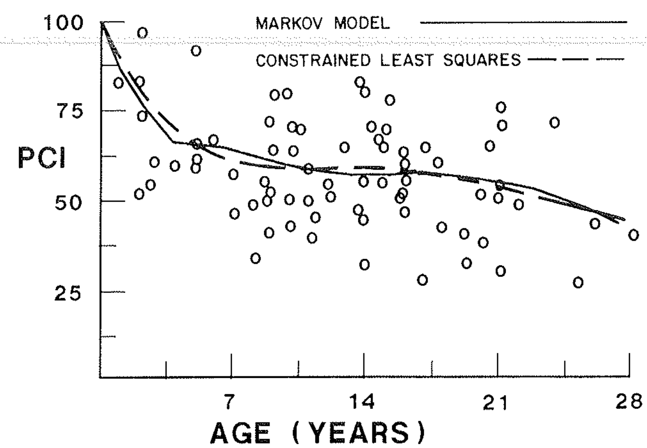

FIGURE 11 Pavement condition prediction curves using new Markov model and the constrained least squares.

Markov model can be used in dynamic programming to produce optimal M&R strategies for the selected pavement sections.

CONCLUSIONS

A pavement-performance and prediction model has been developed that is based on the Pavement Condition Index (PCI) and the age of the pavement. A combination of homogeneous and nonhomogeneous Markov chains has been used in the development of the model.

The Markov model introduces a rational structure to the pavement-deterioration modeling process and is the best for extrapolation. The Markov process will be used in conjunction with the dynamic programming to produce optimal M&R strategies for all the pavement sections in a network. To produce these optimal strategies, the future condition of the pavement is required. Accurate predictions are used for life-cycle cost analysis at the project level and establishing the feasible M&R strategies at the network level. The new Markov model has the

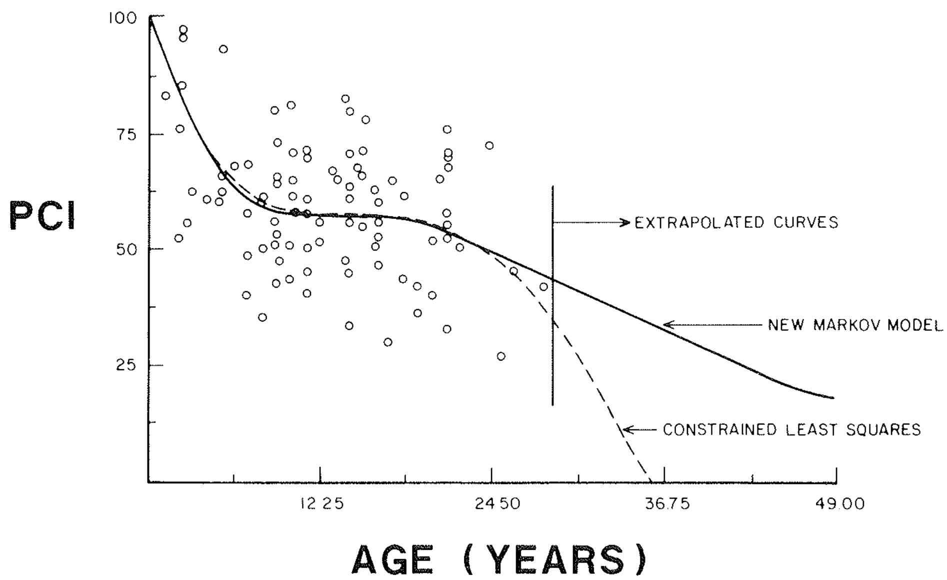

FIGURE 12 Pavement condition extrapolation using the new Markov model and the constrained least squares.

capability of providing this information with minimum effort on the part of the user and with better accuracy and reliability than other techniques that require unsupportable assumptions.

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Roadway Modeling and Data Conversion for a Transportation Facilities Information System

W. K. Bottiger and W. P. Kilareski

Many transportation agencies are in the process of developing roadway management systems to assist with their rehabilitation and maintenance programs. All require a centralized data base to manage the large amount of data collected for each highway section. Described in this paper is a research project that used the utility industry (gas, electric, and so on) interactive graphic data base for highway applications. As a roadway section is similar to an electric line, many of the modeling concepts are therefore also similar. A computer model of the highway system was developed that used five types of facilities. Type 1 is a point facility (sign, signal), whereas Type 2 represents a highway span facility. The Type 4 and 5 facilities represent the data elements and the pictures associated with the interactive graphics. All data are stored at appropriate *X* and *Y* coordinates. The roadway model developed with the five facility types allows the user to trace the connectivity of the highway route as well as to obtain the requested highway information. Also described in this paper is a case study that was conducted to test the model. U.S. Geological Survey maps were digitized and merged with the Pennsylvania Department of Transportation's Systematic Technique to Analyze and Manage Pennsylvania Pavements condition information for roadways in a county in Pennsylvania. It was found that the facility models adequately describe the highway system. The user, however, must be careful in the digitizing process. Map document shrinkage and expansion (due to humidity) is enough to change the document location reference. It was also found that manual digitizing is extremely time consuming, and it is necessary to develop computerized interface data conversion routines.

The highway network in the United States represents a capital investment of more than one trillion dollars. The operation of the network has a direct effect on the social, economic, and political well-being of the nation. For many years the nation's highways have served the public efficiently; today, however, the roadway infrastructure is deteriorating rapidly. Unfortunately, the rate of deterioration is much greater than that at which repairs and rehabilitation can be accomplished. Curtailed revenues, increased construction costs, and increased truck traffic have created a dilemma for highway administrators. Often, the highway engineer has so many projects to undertake, he does not know which one to program first. Consequently, many transportation agencies are developing pavement management systems (PMS) to help manage their highway network.

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The term "pavement management," or "roadway management," describes strategies used at various levels of highway administration. Pavement management systems usually encompass "all activities involved in providing and maintaining pavements at a satisfactory level of service. These activities range from initial data gathering to planning, design, construction, maintenance, rehabilitation, and periodic monitoring of existing pavement condition" *(1, 2)*. A pavement-management system can also provide the information required to determine alternative strategies as well as the optimum treatment required for a particular highway segment. During the past several years many PMS have been developed and implemented at the state and local level. Some PMS are designed for the network level while others concentrate on project-level needs. Although the structure of a PMS can be complex, all PMS use a computerized data base. In fact, all pavement management systems are useless without the data base. Data processing needs range from large mainframes to microcomputer systems.

Highway agencies, especially ones with extensive highway networks, require large amounts of diversified information to manage their systems. This information may contain data about a pavement's history, such as construction records, material specifications, and as-built standards. It may also include the current status of the highway with respect to longitudinal roughness, skid resistance, and distress conditions. Such ancillary information as accident location, culvert location, guiderail status, and bridge data also serve useful purposes.

Some highway agencies have been collecting this inventory and condition information for decades while others have only recently begun to gather information about their system. Because each agency may be responsible for thousands of miles of highway, the problem usually is not whether there are data available, but rather that there is too much information to evaluate. Frequently, data are found at many different locations within an organization. For example, accident information is maintained in the safety unit, truck traffic data in the traffic engineering section, material records in the construction section, and maintenance activities in the maintenance group. Yet the highway administration needs all of this information to make sound decisions.

For a PMS to be effective, the data collected throughout the agency should be integrated both to support analysis applications and to provide for ease of information access for everyday uses. One means of providing this ease of access is through use

of computer graphics as a key to information in the data base. In this scenario, the information available to the highway administration can include not only the descriptive data (structural properties, maintenance history, and so on) but also the location and connectivity (logical relationship of one location to another) of the various facilities that make up the highway and bridge network.

Three major problems result from the volume and diversity of the data required:

1. As data are collected from a number of sources over a period of years, it is difficult to maintain an accurate, up-to-date, and centralized data base.
2. Because of the diversity of input sources, it is difficult to maintain an accurate and complete data base when revisions and updates are required.
3. The large amount of data often makes retrieval of a specific subset of data a time-consuming and tedious task.

These problems may seem insurmountable to a highway agency that has always worked with isolated and independent data sets. However, highway agencies can benefit from the experience of the utility industry, which has a successful history of managing information on geographically distributed facilities, as is discussed in the next section.

UTILITY DATA-BASE MANAGEMENT SYSTEMS

For many years, utility companies (e.g., power, gas, water, telephone) faced the problems of maintaining information about their assets, which are distributed over a large geographic area. Circuit information, pole status, transformer location, and other data were maintained on paper records or maps. Consequently hundreds, and even thousands, of maps and records had to be manually maintained. The problems faced by the utilities were the same as the data-base problems now facing highway agencies. As most utility companies allocate considerable resources to keeping their facility records current and accurate, considerable work went into the solution of these problems. The result of a joint research effort between the utility industry and IBM was the development of the distribution facilities information system (DFIS).

The primary goals of a DFIS are to reduce the cost of maintaining facilities records, to store the records in standard form on a computer data base, and to make the facilities data more readily available in the form best suited to user requirements *(3)*. The DFIS data processing includes two concepts: interactive graphics, which is used to maintain a defined data structure, and a geo-facilities data base.

The interactive-graphics capability is provided by the Graphics Program Generator (GPG) software. GPG is a "set of programs designed to create, maintain, and display information about facilities, their locations and relationships to one another" *(4)*. Through an interactive conversion with the graphics work station, GPG stores data in a structure customized by the using agency. This data structure contains the attribute data for each facility, as well as that facility's location and connectivity to adjacent facilities. The work space developed by one user on a single work station represents a subset of all the agency's data. Once the user has completed the adding or modifying data in his geographic area, the data are transferred to the geo-facilities data base. GPG provides the means for sending data to, and receiving data from, the data base. The maintenance of the geo-facilities data base is provided by the Geo-facilities Data Base Support (GDBS) software. It maintains the hierarchical data model (structure) developed by GPG.

Experience has shown that the majority of data-retrieval requests are by geographic area, and the data model used by GDBS is designed to facilitate such requests. Facilities located near each other in the field are also stored near each other in the data base. An additional component of the data structure permits the connectivity of facilities to be explicitly represented. With this concept incorporated in the data structure, data retrieval for a network can be performed quickly.

The data base maintained by GDBS is continuous. One data base represents the entire area for which an agency is responsible, so there are no breaks introduced by map edges. This centralized data approach ensures that, as data are entered into the system, the information is immediately available to all users. Thus, all data are current and the question of "What data are up-to-date?" can be avoided. The area retrieval capabilities mean that data can be obtained for specific geographic areas, such as political boundaries, service regions, engineering districts, and others. The network-retrieval capabilities, in contrast, permit a user to obtain data for any network of facilities available in the data base.

At the present time the DFIS is being used primarily by utility companies. Consequently, all of the development and implementation work has involved the writing of menus, pointing rules, and so on, for utility applications. As the utility land base and other facilities are similar to transportation facilities, it is logical to manage pavement management system data with GPG. Therefore, the objective of this research was to build the foundation for a transportation facilities information system (TFIS) that will address the issue of collecting and maintaining data on an agency's highway and bridge network.

Whenever interactive graphics is used with a data base the fundamental question of data modeling must be addressed. Too often, little thought is given to the structure of the data base. Consequently, the highway agency is forced to accept a data base that was designed for some other application. The users quickly learn that they must continually modify their operation to make the data base system work, rather than have the data base support their needs. A basic question for PMS is how the roadway system should be modeled in the data base. The research discussed in the paper was designed to create a roadway data base that realistically models the highway system.

ROADWAY MODELING CONVENTIONS IN TFIS

The GPG and GDBS software products allow considerable flexibility in how data can be modeled within a TFIS. This is desirable, as transportation organizations need to manage information that varies widely in form and purpose. Some types of information will rarely need to be updated, including information on the geometry of the highway section, which typically remains the same until major reconstruction is performed. Other types may need to be updated yearly or even more often.

There are five facility- and data-modeling options available within the framework of the GPG/GDBS system. These conventions are named Type 1 through Type 5, and each has a

specific data-modeling purpose. This discussion will outline each of the conventions and describe what facilities and data records in a highway and bridge network have been modeled using each convention, as well as provide a description of the attribute data associated with that facility.

It should be noted that this discussion involves only those facilities that were developed specifically for the TFIS by the project team. It does not cover such items as work entities or the entities used for the plotting of data. The facilities discussed are specifically those that would be of interest to a transportation organization implementing TFIS, and are items that are expected to be found as parts of the physical highway and bridge network.

Type 1 Facilities

Type 1 facilities are those that exist at a single set of *X*, *Y* coordinates and are attached to a single point connector at that location. Point connectors are defined as a method of modeling explicit expressions of connectivity between facilities at the same location *(5)*. The logical data model associated with Type 1 facilities is shown in Figure 1. The Type 1 convention is typically used to model facilities that have the ability to stand alone at a single physical location. Type 1 facilities do not have the ability to control connectivity across a single *X*, *Y* location. In a utilities application, items that are typically modeled as Type 1 facilities include poles, manholes, landmarks, and lot and block locations of utility service customers.

Type 1 facilities may also have subfacilities or repeating data groups associated with them. A subfacility (Type 4) is a facility that is subordinate to a Type 1, 2, or 3 facility in the data hierarchy and that has a picture associated with it. A repeating data group (Type 5) does not have a picture associated with it, but is capable of storing a number of records of the same format, typically of a periodic nature. The example commonly used in utilities applications is the situation in which a power pole is modeled as a Type 1 facility, has a guy wire (a subfacility of the pole) modeled as Type 4, and has a series of yearly inspection records for the pole (a repeating data group) modeled as Type 5. The facilities that were modeled as Type 1 for highway application include: intersection, culvert, sign, signal, and railroad crossing.

Type 2 Facilities

Type 2 facilities are those that exist between two sets of *X*, *Y* coordinates and are attached to a point connector at each end. The logical data model associated with Type 2 facilities is shown in Figure 2. The Type 2 convention is typically used to model facilities that make up the network and for which connectivity information must be maintained. A Type 2 facility is typically any facility that can be represented as a line, straight or otherwise, between two points. As most facilities of interest to transportation organizations are of the span type, this is a heavily used convention in TFIS. In the utilities application area, items that are typically modeled as Type 2 facilities include pipes, primary and secondary circuits, and property lines. Type 2 facilities may also have subfacilities or repeating data groups associated with them, as shown in Figure 2. It should be noted that most of the Type 4 and Type 5 facilities used in TFIS are subordinate to Type 2 facilities. Subfacilities and repeating data groups are described in detail in the following sections on Type 4 and Type 5 facilities. The facilities that were modeled as Type 2 for highway application include: highway, tunnel, bridge, railroad, river, and boundary.

Type 3 Facilities

Type 3 facilities are those that exist at a single set of *X* coordinates and are attached to two point connectors at that location. The logical data model associated with Type 3 facilities is shown in Figure 3. In utilities applications of GPG/GDBS, Type 3 facilities are typically used to model those facilities that control flow through the network, whether it be

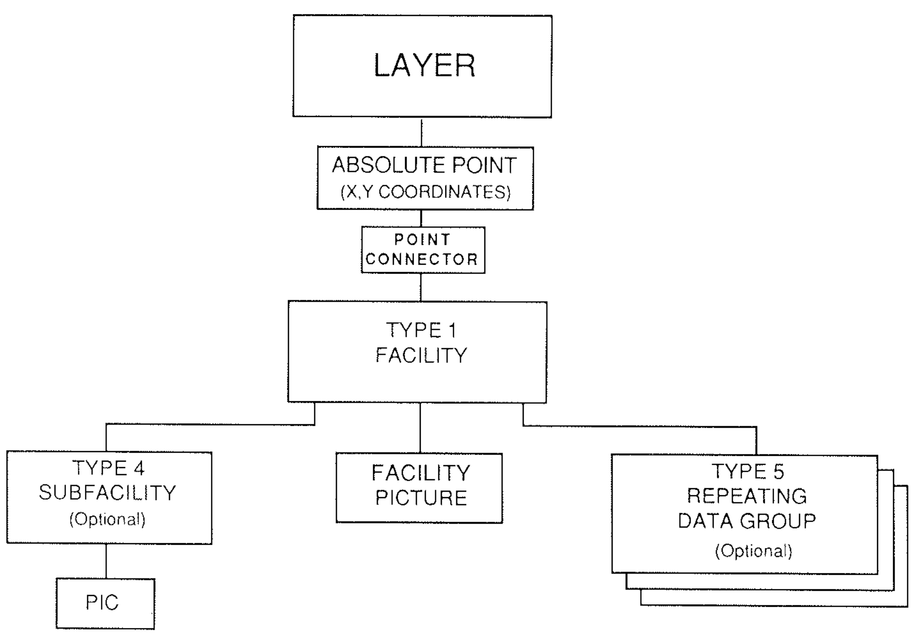

FIGURE 1 Data model for Type 1 facility, point facility.

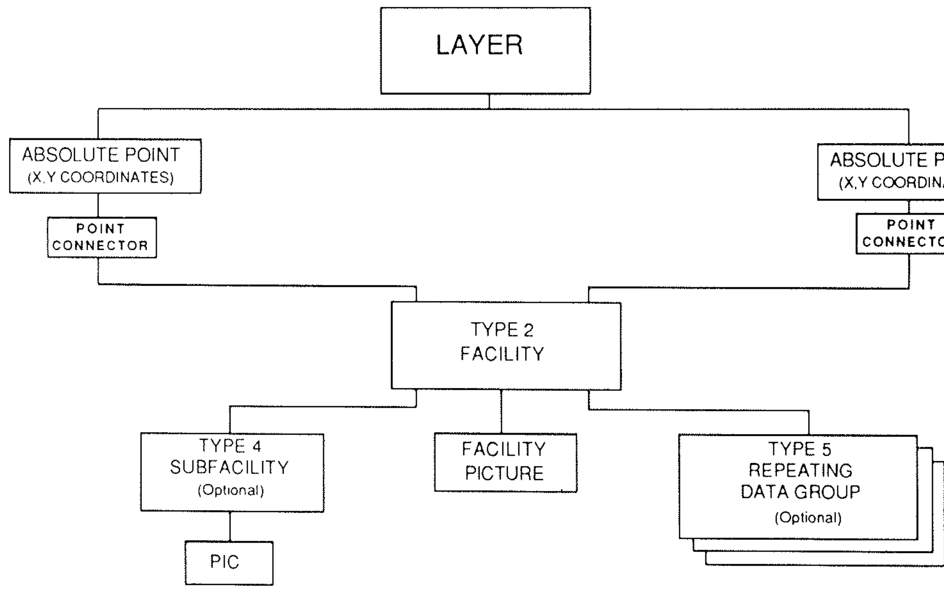

FIGURE 2 Data model for Type 2 facility, span facility.

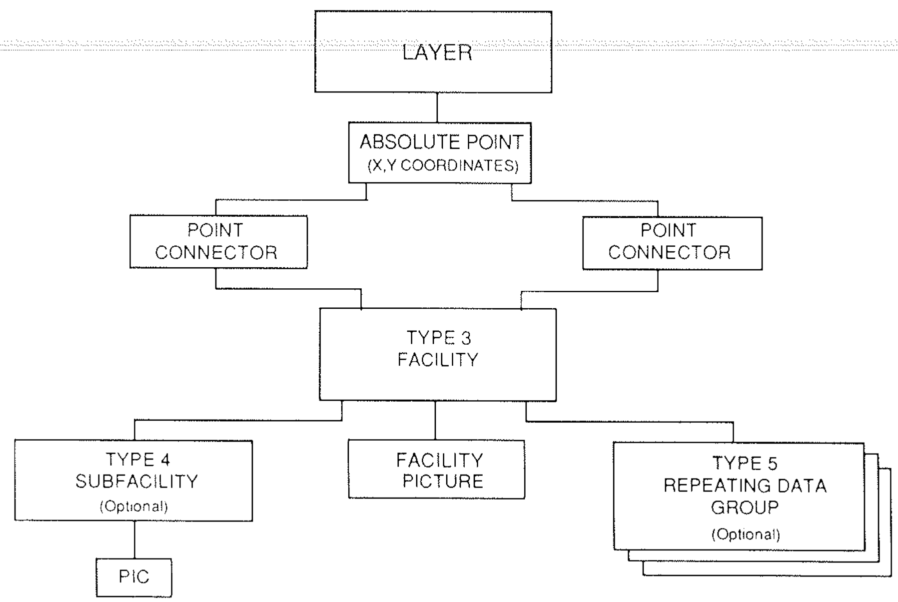

FIGURE 3 Data model for Type 3 facility, connector.

electricity, water, or natural gas. Such items as values, transformers, and switches are normally modeled as Type 3. In general, Type 3 facilities are useful for modeling items that control some type of flow across a point.

No use for the Type 3 facility has been found in the TFIS project to date. Although some consideration was given to the possibility of modeling intersections as Type 3 facilities, it was discovered that the subroutine that expands the centerline on highway facilities (draws curb lines parallel to the centerline at one-half the width) would not work properly if intersections were modeled as Type 3. It was subsequently decided simply to attach all of the approaches to an intersection to a single point connector, and model the intersection as a Type 1 facility attached to the same point connector. An organization implementing TFIS should, of course, evaluate its particular data-management needs to see if any facilities should be modeled as Type 3.

Type 4 Facilities

Type 4 facilities are typically subfacilities of facilities modeled as Type 1, 2, or 3. Thus they are subordinate to their parent facilities, and if the parent facility is deleted from the work space, the subfacility is also deleted, as it is lower on the data hierarchy than the parent facility. Type 4 facilities can be accessed by scanning the data hierarchy; that is, it is possible to look for all subfacilities associated with a particular facility, and process them en masse. Since Type 4 facilities also have pictures associated with them, it is also possible to access a particular Type 4 facility by pointing to its picture on the graphics screen or map.

Type 4 facilities (subfacilities) may also have Type 4 or Type 5 facilities associated with them lower on the data hierarchy. This ability to nest subfacilities and repeating data groups allows a considerable volume of information to be associated

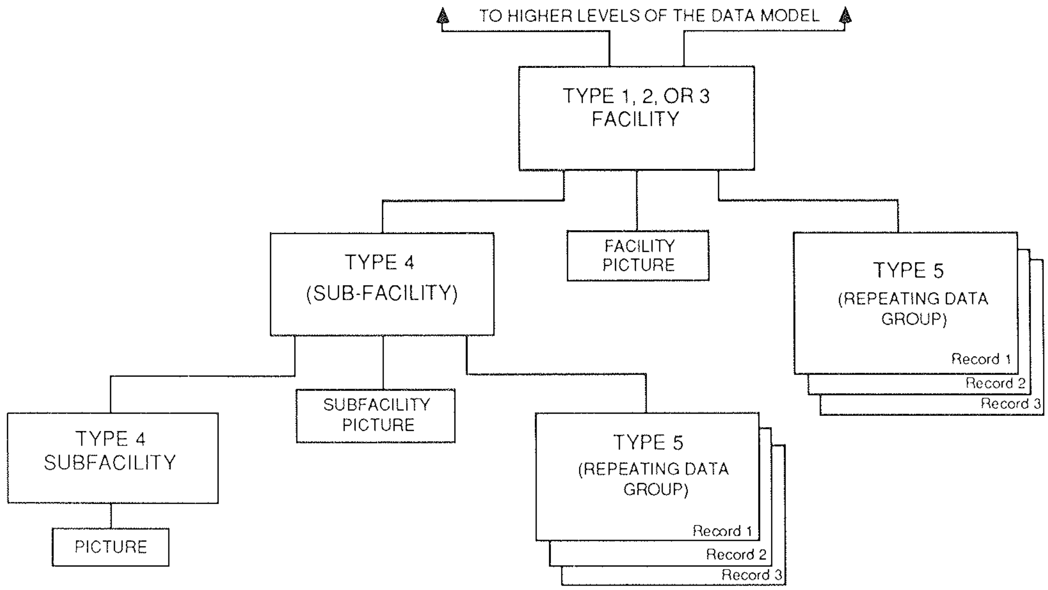

FIGURE 4 Nesting of information in the data model.

with a given facility in the work space. The concept of nesting in the data structure is illustrated in Figure 4.

In utilities applications (DFIS), several examples of the use of Type 4 facilities are found. Perhaps the best is the modeling of a transmission pole guy wire as a subfacility of the transmission pole. If the pole (Type 1) is deleted from the work space, it is not desirable to have a guy wire facility in the work space with "nothing to guy." This problem is alleviated by modeling the guy wire as a Type 4 so that it is deleted from the work space if the pole is deleted.

In the TFIS, the following entities have been modeled as Type 4 facilities: manual patching, shoulder cutting, pipe replacement, and surface treatment. All routine maintenance activities are modeled as a Type 4 facility as the maintenance treatment is a subfacility of the highway section (Type 2) or the point facility (Type 1).

Type 5 Facilities

Type 5 facilities are referred to as repeating data groups. They are subordinate in the data structure to Types 1, 2, and 3 and can also be subordinate to Type 4 subfacilities. Repeating data groups are especially useful in situations where periodic records must be kept on a particular facility or subfacility. Such items as the periodic inspection of transmission poles for signs of deterioration are likely candidates for modeling as Type 5 in utilities applications. A repeating data group record may be repeated any number of times for a particular facility. The Type 5 facilities have no pictures associated with them, as do the Type 4 facilities.

With reference to a roadway management system, the Type 5 facilites are where the majority of data elements are stored. For example, geometry is a repeating data group. The group contains information on the geometry and alignment of a highway section. The following data fields are part of geometry: length, grade, shoulder type, shoulder width, median type, median width, control areas, degree of curves, and superelevation. Each Type 5 facility can have almost unlimited data fields. The other Type 5 facilities developed in this project include: traffic, pavement design, safety, pavement monitoring, and programmed maintenance.

DEALING WITH ROADWAY-MANAGEMENT SECTIONS IN TFIS

After the framework for the TFIS was developed and the categories defined for the facilities, a case study was performed to determine the feasibility of the system. In this scenario, the code developed was tested on a highway network involving part of Centre County, Pennsylvania. Specifically, the sample application included the area covered by the State College and Julian, Pennsylvania, 7.5-minute U.S. Geological Survey (USGS) topographic quadrangle maps (see Figure 5). It was expected that, by using an existing highway network structure, deficiencies in the developed software control files would be more quickly uncovered than if the code were tested on hypothetical networks.

These expectations have indeed been borne out, and many problems or potential problems have been discovered in the data-modeling and programming conventions. Although not all of these problems have been solved, their discovery has at least given the project team a chance to document them, so that a user implementing TFIS will not make the same mistakes or need to duplicate the efforts of the project team.

One critical data-modeling aspect that has come to light in the Centre County sample application is the need to be able to logically unify a series of digitized highway facilities into a single, homogeneous entity known as a roadway management section (RMS). An RMS is defined by most highway agencies as a section of highway having a defined beginning and ending point and homogeneous properties throughout its length of 500 to 2,500 ft. These properties may include pavement type, pavement roughness and degree of deterioration, geometry, amount and distribution of traffic, number of accidents, and numerous other items. The end points of the RMS are defined in terms of *X*, *Y* state plane coordinates.

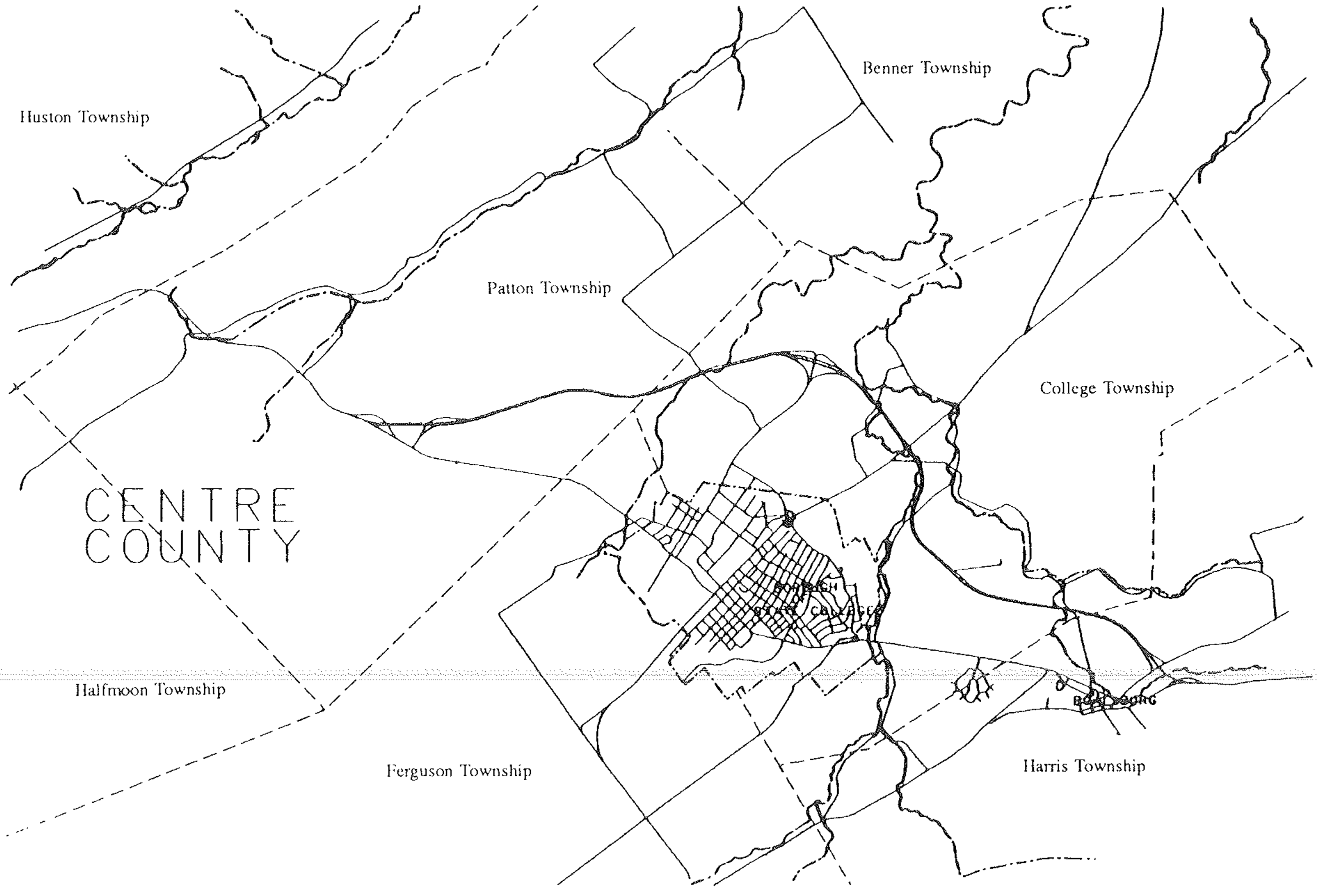

FIGURE 5 Centre County TFIS sample application areas.

Digitized Highway Sections Versus Roadway-Management Sections

Information on roadway-management sections in the Centre County application was derived from a number of sources provided by the Pennsylvania Department of Transportation (PennDOT). These included STAMPP microcomputer data files, county maps showing legislative route (LR) numbers, and straight-line diagrams depicting stationing of intersections and various other objects and points of interest.

Using these available information sources, the RMSs were scaled on the map from locations of known station (such as an intersection or stream crossing), and their ends marked and noted on the map along with such information as the beginning and ending station of the RMS. Stations and physical (map) locations of items such as bridges and culverts were also noted on the map documents (Figure 6). Having been digitized into the work space, the facilities were then edited using GPG's full-screen editor. Descriptive data derived from the STAMPP data files and the straight-line diagrams were added to the attribute fields in this process. An example of the attribute editing screen is shown in Figure 7.

At this point in the digitizing process, each Type 2 span facility digitized corresponds to a roadway-management section. In other words, there are no breaks in the facility between the end points of the RMS, and each is continuous. An example of a roadway management section is shown in Figure 8. The RMS is the section of South Atherton Street stretching between the College Avenue and Hamilton Avenue intersections indicated by shading. It begins at station 0 + 00 and ends at 26 + 74 and is 2,674 ft long.

Difficulty with the data model begins to arise, though, when side streets and highways that connect to the RMS are digitized into the work space. The GPG pointing rules used to add highway sections are written so that if a highway facility already exists at the point where the new facility is to be added, the software splits the existing facility and adds a new absolute *X*, *Y* point at that spot. The reason for this is to maintain the connectivity of the network, so that if a highway network trace is desired at some time in the future, there will be connectivity in the data base between the RMS and the side street, as there is in reality. As soon as these splits begin to occur, the RMS becomes partitioned and fails to be the single, continuous entity it was when originally digitized.

Thus, when side streets connecting South Atherton Street were digitized, in our example splits occurred at each new intersection that was created. This means that splits occurred at the intersections with Beaver, Foster, Nittany, Fairmount, and Prospect Avenues, and the original single RMS section was therefore partitioned into six separate and distinct highway sections. The only thing that bonds them together is commonality of the attribute data fields; when the original RMS was split up, all of the subsections were given data fields that were carbon copies of the original. As the situation stands at this point, each of the six highway sections that make up the example RMS can have its attribute data edited independently. This means that even though it may not be intentional on the part of the user, different facilities making up the RMS may have different attributes, a situation that violates the assumption of homogeneity of the RMS.

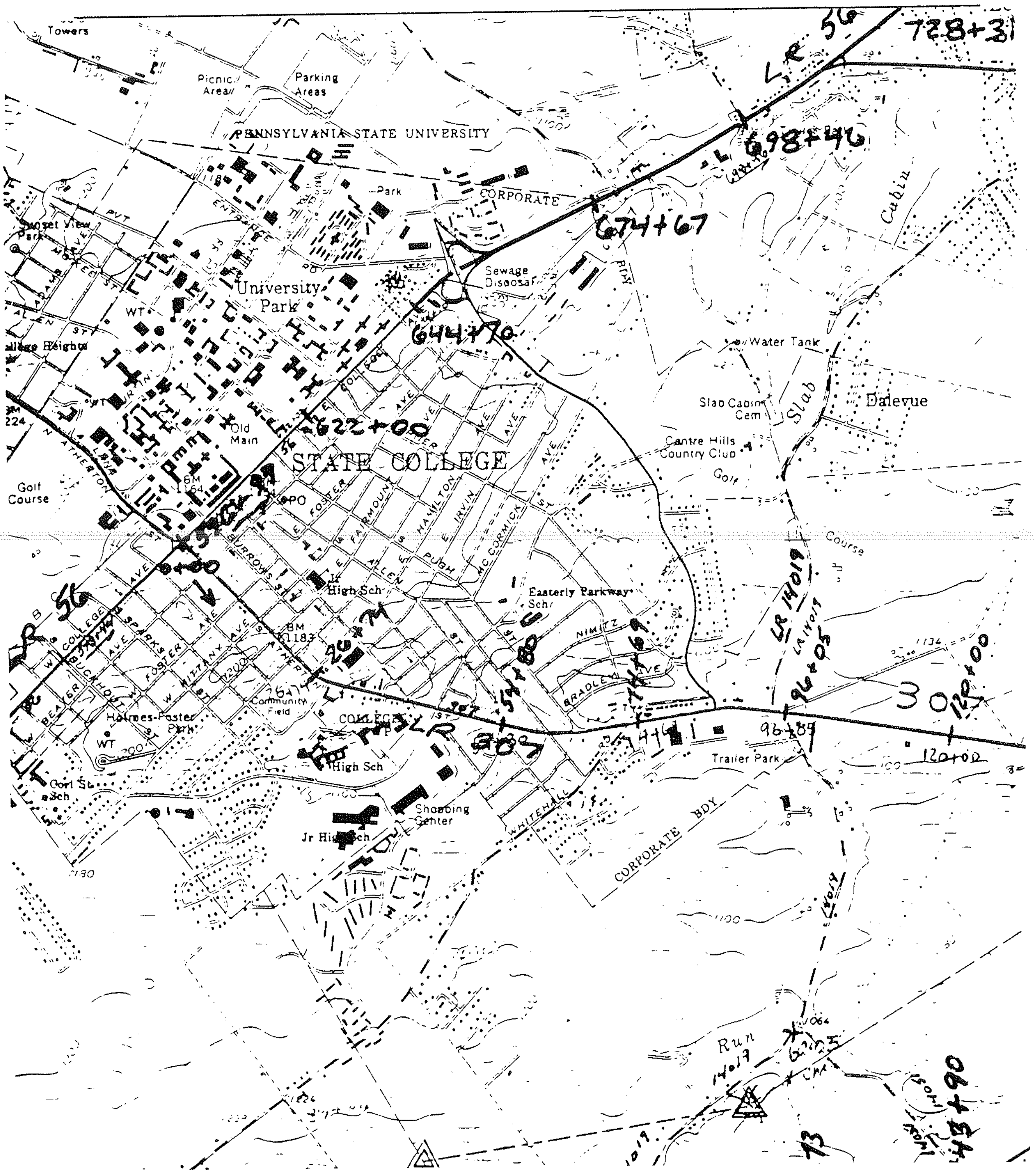

FIGURE 6 Section of map document with scaled RMSs and annotation.

From the foregoing discussion, it is apparent that some thought must be given by an agency implementing TFIS to the method of modeling a roadway-management section. Some method needs to be devised by which a series of highway facilities in a TFIS work space can be logically glued together into an RMS.

It was seen fairly early in the research project that perhaps the glue might best manifest itself in the form of an RMS identifier code that would be included as part of the attribute data of each highway facility. For this reason, the facility definition for highways, which defines all of the attribute information associated with a highway, contains the field "ID", an 8-byte character data field.

This ID field was defined for character data in case the organization implementing TFIS uses some form other than an integer numbering system to define section identifiers. The use of character data allows some imagination to be used in designing RMS identifier codes. In some sections of the Centre

```
EDIT HIGHWAY     210   TYPE: 2   LAYER: E KEY: SYS      11     FIELD 1     MAX 24
 PT1:     1968436       227357        PC1:    1
 PT2:     1968855       226935        PC2:    1                  CASE   UPPER
-----------------------FIELD DATA-----------------------------------------
 ID       NAME      SUB   TYPE          CHG?          VALUE
 1      USERID            CHAR       E   -     BRENDA
 2      DATE              CHAR       E   -     05/14/86
 3      TIME              CHAR           -     13:07:27
 4      NAME              CHAR           -     SOUTH ATHERTON ST.
 5      WIDTH             REAL           -     24.0
 6      ID                CHAR           -     001LR307
 7      START             REAL           -     0.0
 8      END               REAL           -     26.74
 9      LENGTH            REAL           -     2674.0
 10     DIRN              CHAR           -     EAST
 11     JURSDCTN          ALIAS          -     STATE
 12     NBRLANES          HALF           -     3
 13     PVMTTYPE          CHAR           -     CONCRETE
 14     CURPSI            REAL           -     4.1
 15     LASTMAIN          CHAR           -       /  /
ENTER COMMAND OR PFKEY (PF1 OR HELP FOR INFORMATION)
====> COPY 1
```

FIGURE 7 Example of data fields for a highway section.

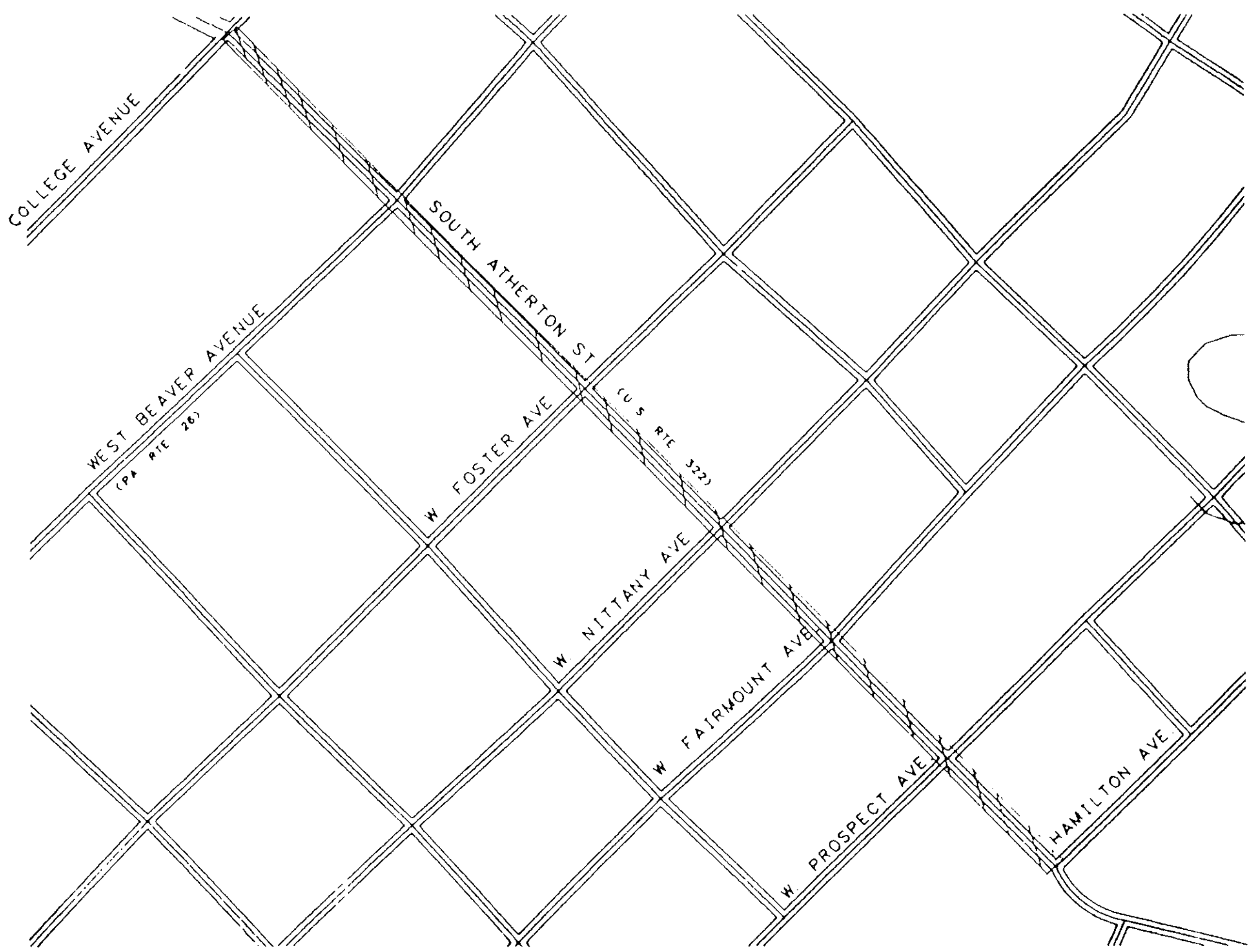

FIGURE 8 Roadway management section after digitizing process.

County sample application, for instance, RMS were given codes such as '001LR307', '002LR307', and so on. Thus, information can be captured in the code to indicate the LR number and the section number within that LR. In the foregoing example, the codes would designate the first and second roadway-management sections within LR-307, which is South Atherton Street (US-322).

The software control files written to deal with roadway management sections use the fact that all highway facilities making up the RMS will necessarily have the same ID code. When a facility is found whose ID matches the one sought, then that facility is flagged, generally by setting one of the bits in the 32-bit status word assigned to each facility.

Once this flagging process has taken place, the facilities composing the RMS are defined, and subsequent processing is generally in the form of a mass update of the attribute data associated with facilities in the RMS. However, the user may merely want a graphic depiction of the extent of a particular RMS. In this case all those sections are made to blink on the graphics screen (although other graphic report options exist).

OTHER CONSIDERATIONS FOR DATA MODELING

The Argument for Digitizing Largest to Smallest

Considerable time and effort can be saved during the digitizing process by giving some thought to the order in which highway facilities are digitized. The Centre County sample application showed that it was most efficient to digitize from the top down. That is, important arterials and Interstate highways were digitized first, then less important LRs for which STAMPP data were available, and last, collector routes and streets.

This digitizing hierarchy stems from the idea of partitioning roadway management sections. A major objective in a data conversion exercise such as digitizing a map is to convert the most data with the least time and effort and to make digitized sections as long as possible. For example, when a 2,500-ft-long roadway-management section is digitized as a single facility, its attribute data need only be edited once in order to transfer the information from a map or the STAMPP data file into the TFIS work space. If, on the other hand, roads of lesser magnitude that intersect the RMS are digitized first, then the same RMS will have to be digitized as a series of facilities and individually edited.

Instability of Map Documents

It has been known for many years that paper map documents are subject to expansion and contraction as a result of changes in temperature and humidity. For most map applications, the degree of change in the dimensions of the document is not noticeable or detrimental. When maps are digitized into a computer data base, however, even small deviations in the scale of the map are quite noticeable given the exceptional precision of modern electromagnetic digitizing equipment.

This fact became readily apparent during the course of the TFIS sample application. Part of the highway network was digitized into the work space each day. With weather and humidity changes from day to day and throughout the day, the paper map documents shrank and swelled, making it difficult to correlate on the parts of the network already digitized into the work space. At one point, correlation on an already-digitized facility was missed by nearly 100 data base units (feet, in this case) when part of the work space was digitized on one day, and then the map was reregistered with the coordinate system on the following day. It is important to note that a change in any dimension of the map of only 0.025 in. is sufficient to cause an error of 50 ft in the work space at this scale (1:24,000).

An attempt to alleviate this problem was made by laminating the map sheets between layers of polyethylene plastic. The idea was that by bonding the paper between layers of a material with different thermophysical properties, the shrinkage and swelling problems would be avoided. While this approach did indeed reduce their severity, it did not totally alleviate them. It is recommended that an organization implementing TFIS copy its map documents onto some stable base material, such as Mylar drafting film, using a photographic process before digitizing from them. Another approach would be to use a larger-scale map so that the size of the errors caused by shrinking and swelling would be smaller in relation to the data base unit.

Precision of Registration Coordinates

Part of the problem in obtaining proper correlation on a paper map document was also found to be the manner in which the state plane coordinates for the registration points on the map corners were originally obtained. Initially, the coordinates for the map corners were scaled from the 10,000-ft grid ticks that the USGS supplied on the map sheets.

It was later realized that some of the registration error could be alleviated by mathematically calculating the coordinates from the latitude and longitude at the corners of the sheet, which have even numerical values. As these are exact values, precise values of the state plane coordinates for the corners can be computed. These values can then be rounded to the nearest foot and input to GPG using the X, Y map registration process. The computation was facilitated by writing an interactive FORTRAN program that allows input of latitude and longitude and computes values for the state plane coordinates. This program makes use of a data file containing 11 parameters for the Lambert polyconic projection for the particular zone (in this case, the Pennsylvania North Zone).

Time Component of Digitizing Maps

Graphic data input into TFIS in the sample application was by hand-digitizing of maps through the use of an electronic digitizing tablet and cross-hair cursor. Attribute data entry was carried out via keyboard, and required individual editing of hand-digitized transportation facility sections. This proved to be an extremely time-consuming and labor-intensive process. The sample application area, consisting of two USGS 7.5-min topographic quadrangles, took a total of approximately 200 man-hours to be digitized, have attribute data transferred from other sources, and be checked for errors. If it is assumed that the typical map sheet will require an average of 100 man-hours to be entered into the work space (graphic data, attributes, and checking for errors), and if a man-hour costs $10 to the implementing organization, then total cost in labor alone to develop a

transportation data base for a state the size of Pennsylvania will be on the order of $1 million.

As most organizations concerned with the management of transportation facilities information already have some type of machine-readable data files, it would probably be more cost effective to put the data files directly into the TFIS data base. The conversion program itself would consist largely of input and output operations: reading the data records from the in-house file and reformatting the data under the conventions of Interface Format. It is quite probable that certain clean-up operations, particularly on the graphics, would need to be performed once the data had been transferred to the GPG work space, but it is expected that the total savings in data conversion time and effort would far outweigh the additional burden of these clean-up operations.

FINDINGS

The modeling of a roadway-management system can be based on previous work done in the utility area. Facilities with attributes distributed over a geographic area are similar in concept. Electric transmission lines and roadway segments can be similarly modeled. A TFIS model can be made up of several types of facilities. Type 1 represents point facilities, whereas Type 2 represents span facilities. The Type 4 and 5 facilities represent the data elements associated with the roadway network.

The application of TFIS to an existing highway and bridge network made apparent some of the problems that may occur when an organization responsible for the maintenance of information on transportation facilities implements TFIS in a production environment. Specifically, these problems include the instability of paper map documents, the precision of coordinates used when registering map documents, and the time and cost involved in digitizing maps.

Based on the problems encountered in the Centre County application of TFIS, which were previously discussed, the following recommendations are made to any organization implementing a TFIS.

1. Map documents to be used in the digitizing process should be copied onto a stable medium that is not as subject to the problems of expansion and contraction as paper.
2. Coordinates of points used for the registration of the map document on the digitizing tablet should be determined with a precision of not less than the data base unit (typically 1 ft). In most cases, this will preclude the method of scaling coordinates from the map document itself. Coordinates of sufficient precision may be computed mathematically from true spherical coordinates noted on the map document (such as latitude and longitude values noted on USGS maps).
3. The possibility of converting machine-readable data files already existing in the organization into a format usable with the TFIS software should be considered. It is quite possible that the programming effort involved in the conversation will be far more cost effective than hand-digitizing of map documents and the subsequent manual entry of attribute data. Moreover, this method offers less chance for human error to cause problems with data integrity.

The research team has determined that if sufficient forethought and planning are allocated to these issues, TFIS can indeed be successfully applied to manage information on geographically dispersed transportation facilities. Further research and development efforts on this highly flexible system are likely to yield a system that is more efficient, cost effective, and easy to use, and that will in the long run greatly reduce the information-management costs of organizations charged with responsibility for transportation facilities.

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Development of a Methodology to Estimate Pavement Maintenance and Repair Costs for Different Ranges of Pavement Condition Index

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This paper presents a network-level procedure for determining the best maintenance and repair alternative and its associated cost for different pavement categories at different Pavement Condition Index ranges. Data from a number of military installations in the United States were used, and the analysis was performed separately for each installation. The methodology developed included techniques for *(a)* Determining the fixed initial construction cost of each alternative based on local prices; *(b)* Determining the cost of pavement preparation before repair as a function of pavement type, condition, local prices, and installation policy for pavement preparation; *(c)* Determining the annual cost of routine maintenance of each maintenance and repair alternative as a function of pavement condition, local prices, and installation maintenance policy; *(d)* Determining pavement performance characteristics (service life and rate of serviceability deterioration) for various pavement categories; and *(e)* Conducting a life-cycle cost analysis of each alternative for all pavement categories at various Pavement Condition Index ranges using the equivalent uniform annual cost approach.

After several decades of adequate service, pavements on military installations, like those of the rest of the other highway systems, are deteriorating at a fast rate. In recent years, maintenance and repair activities have not been able to keep pace with the rate of deterioration of highway pavements. This impending infrastructure crisis has confronted military pavement engineers with questions for which they have no ready or documented answers. The difficulty of assessing maintenance and repair needs, budget requirements, maintenance and repair alternatives and their cost-effectiveness, has resulted in the development of a systematic pavement management system (PAVER) by the U.S. Army Corps of Engineers *(1)*.

The PAVER system consists of a computerized data base and a number of programs that store, retrieve, and manipulate data, as well as perform a variety of analyses and calculations required for network and project-level decisions. PAVER's capabilities include: *(a)* data storage and retrieval, *(b)* pavement network identification, *(c)* pavement condition rating, *(d)* project priority setting, *(e)* inspection scheduling, *(f)* maintenance and repair needs determination, *(g)* resource planning, and *(h)* economic analysis and budget planning.

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The Pavement Condition Index (PCI) is the basis for the PAVER pavement management system. The PCI is a scale from 0 to 100, with 100 being excellent, and is determined based on measured distress type, severity, and amount.

The PAVER system was developed to assist installation engineers and planners with pavement management by providing an extensive data base and valuable computational and report-generating capabilities. One of its most useful and widely used network-level planning programs is its budget-planning report, or BUDPLAN. The execution of BUDPLAN and a number of other programs requires the user to input area unit costs for maintenance and repair alternatives at various pavement conditions (PCI ranges). Based on predicted pavement condition and input unit costs, PAVER computes a 5-year maintenance and repair budget. These estimates can then be used to justify present and future funding requests.

The estimation of unit costs for maintenance and repair activities at various PCI values requires that the user be familiar with the PAVER system and have complete maintenance and repair records. As the PAVER system is only now being implemented at many military installations, or at most has been on-line for a few years, it is doubtful that system users can generate valid cost estimates. Furthermore, an error in unit cost data, in relation to pavement condition, can result in erroneous estimates of budget needs.

The overall objective of this research project was to develop a rational procedure by which unit maintenance and repair costs at a given installation could be estimated as a function of pavement condition. Based on results from this study, average square yard costs for different pavement categories at various pavement condition ranges can be incorporated into the PAVER system or used as guidelines by PAVER users.

STUDY APPROACH

In order to develop reasonable cost estimates and relate them to the PCI levels, several tasks were performed, as discussed below.

1. Development of a comprehensive data base that includes all necessary information. This was done through the modification

and screening of the PAVER data bases available through the Construction Engineering Research Laboratory (CERL) of the U.S. Army Corps of Engineers.

2. Grouping of pavements into classes based on construction type and traffic levels.
3. Grouping of maintenance and repair alternatives into a number of discrete activities, which were: annual routine maintenance, surface treatment, thin overlay, thick overlay, and reconstruction.
4. Grouping of PCI values into ranges (0–20, 20–40, 40–60, 60–80, and 80–100).
5. Analysis of life-cycle costs for each pavement class to determine the most cost-effective maintenance and repair alternative for each PCI range.
6. Development of a relationship between the PCI and maintenance and repair costs for each pavement class.

In the remainder of this paper, each of these tasks is described in detail. Results from different military installations are also presented as an example.

DATA BASE DEVELOPMENT

The main source of the data used in this research was the PAVER data bases made available through CERL. However, several modifications were carried out to reduce and screen the available data to a form suitable for the objectives of this project. The data base included detailed information from five military installations (Fort Eustis, Fort Knox, Great Lakes, Sierra Army Facility, and Tulsa) and consisted of 2,517 records. Each record included the following main categories of information:

1. Section identification

- Military installation code
- Inspection number
- Section length
- Other items

2. Pavement rank (traffic category)
3. Pavement structure

- Surface type, thickness, and date of construction
- Base type, thickness, and date of construction
- Other items

4. Pavement condition

- Inspection date
- Amount and severity level of each distress type and associated deduct points
- Overall PCI

PAVEMENT CLASSIFICATION

Pavement sections were grouped based on structure type and traffic level. Although initially it was found that there were 14 pavement structure types, it was decided to group them into four major categories: *(a)* asphalt concrete, *(b)* surface treatment, *(c)* thin overlay (less than 2 in.), and *(d)* structural overlay (more than 2 in.). Traffic level was also considered through grouping pavement sections based on their rank. Three basic pavement ranks are used in the PAVER system: primary, secondary, and tertiary, with primary being the rank with highest traffic level. Thus, pavement sections were grouped into 12 classes (four pavement structure types and three pavement ranks or traffic levels).

It should be noted that this study was limited to nonfamily, asphalt roadways only. Results may not be applicable to parking lots, airfields, or rigid and asphalt-overlaid rigid pavement. However, the methodology described in this paper can be used to develop similar results for any pavement type.

PCI RANGES

Since the objective of this research was to develop relationships between unit costs and pavement condition as defined by the PCI, it was necessary to establish the PCI ranges for which unit cost information was to be developed. To comply with the BUDPLAN report's input requirements, it was decided to use the following five PCI ranges:

PCI 81–100
PCI 61–80
PCI 41–60
PCI 21–40
PCI 0–20

MAINTENANCE AND REPAIR ACTIVITIES

In selecting maintenance and repair (M&R) activities to be included in this study, two items were considered. First, the selected M&R actions were comparable to those listed in the available data base, otherwise it would have been impossible to have obtained performance and cost information on any of the activities. Second, general groups of these maintenance and repair activities were included, rather than specific project-level activities, because the research was conducted at the network level. The following maintenance and repair actions were found to be common to all installations:

Annual maintenance only
Surface treatment
Thin overlay (< 2.0 in.)
Structural overlay (> 2.0 in.)
Reconstruction

It should be noted that in some cases reconstruction includes both the base and surface courses, while in other cases reconstruction includes only the surface course. Furthermore, although recycling was initially included as an option, installations included in this study do not consider it to be cost-effective for small-scale rehabilitation projects. Discussions with installation engineers and reviews of past contract documents indicated that each installation's definition of various M&R actions and what they consist of was somewhat different. Therefore, it was necessary that unit cost estimates be derived separately from the work items that are commonly included in each M&R alternative at each installation. As the work items for a particular M&R alternative are different at different installations, a weighted average approach was used to estimate

unit activity costs by considering the percentage of times a particular work item was included in the data on the number of projects for a particular M&R alternative in an installation.

SELECTION OF MOST COST-EFFECTIVE MAINTENANCE AND REPAIR ALTERNATIVE

The purpose of this section is to illustrate the procedure used to select the most cost-effective maintenance and repair alternative. The methodology is based on a comparison of alternatives using a life-cycle costing procedure. Life-cycle costing was based on both the cost and performance of each alternative.

Maintenance and Repair Unit Costs

To estimate the life-cycle cost of any alternative, both its service life and its unit cost must be known. Unit costs associated with each repair alternative included initial cost and routine or annual maintenance costs during the service life of the alternative. User costs were not considered, because the role of user costs on low volume military roads is not well established. Furthermore, since the results of this project will be used for budget estimates, results in terms of agency costs only are relevant.

Initial Costs

Initial costs of any M&R alternative are made up of both a fixed-cost component and a variable-cost component. The variable-cost component depends on the amount of pavement preparation required. The methodology used to determine both components is described below.

Fixed Initial Cost

The fixed initial cost of an M&R alternative is a function of both the local prices and the physical layout of the installation's highway system. The total square-yard fixed-unit cost for each maintenance and repair alternative was calculated using the following simple cost formula:

$$T_k = \sum_{i=1}^{n} C_{ik} * F_{ik} \qquad (1)$$

where

T_k = total square yard fixed cost for the kth M&R alternative,
C_{ik} = average square yard unit cost for the ith cost item used in the kth M&R alternative,
F_{ik} = frequency of use of the ith cost item in the kth M&R alternative, and
n = total number of cost items.

Various cost items are not uniformly used every time an activity is undertaken. Unit costs along with frequencies of use of different cost items were obtained through field visits to different installations where key project information such as the project specifications, quantity estimates, and actual bid abstracts were reviewed. The frequency of use of a cost item for a specific M&R alternative was determined by dividing the number of times an item was used by the total number of projects in this alternative. The frequency of use factor was used to reflect the degree of use of different cost items, which may vary significantly from location to location.

Cost of Pavement Surface Preparation

The second component of any M&R alternative's initial cost is the expense associated with pavement preparation before the application of the M&R alternative. Pavement preparation cost depends on two key factors: *(a)* pavement condition at repair time, and *(b)* local repair policy that determines what surface preparation is to be done before executing a specific repair activity.

Surface preparation cost was related to PCI level through the use of the distress density matrix after the identification of each installation's surface preparation policy. Distress density is defined as the percent of section area indicating a specific distress type and severity level. The density matrix of a specific pavement class summarizes the average density values for each PCI range by distress type-severity level combination. In this project a density matrix was developed for each pavement class within the five military installations. An example of the density matrix is presented in Table 1.

An installation's surface preparation policy was obtained through interviews with facility engineers. From these interviews, both the installation policy in terms of actions taken to prepare pavement surface before repair and the associated unit cost were obtained. For example, considering the average of all installations, it was found that pavements with high-severity alligator cracking are usually maintained or the surface prepared with deep patches at an average cost of $3.60/yd^2. A surface preparation policy was identified for each installation to indicate the action and associated unit cost for different distress type-severity conditions. An example is shown in Table 2.

Calculation of Surface Preparation Costs

The average density values obtained from the density matrix were combined with the installation surface preparation policy to arrive at a total surface preparation cost by PCI range as follows:

$$PC_{kl} = .09 - \sum_{i=1}^{19} \sum_{j=1}^{3} * D_{ij} * C_{ij} \qquad (2)$$

where

PC_{kl} = total surface preparation cost for the kth surface type at the lth PCI range;
i = distress type (1, . . . , 19);
j = distress severity levels (1, 2, 3);
D_{ij} = average density (percent) of the ith distress type with jth severity-level combination for a PCI range;

C_{ij} = unit cost of surface preparation required for the ith distress type with jth severity-level combination; and

.09 = constant to convert ft^2 to yd^2 costs and to change density from a percent value to a ratio.

A sample calculation is illustrated in Table 3. Assume a pavement has only three distress type-severity level combinations for a PCI range of 61 to 80, and that unit surface preparation costs are as shown.

Determining Total Initial Costs

Finally, total initial cost (fixed + surface preparation) was calculated for each M&R alternative for all pavement classes by PCI range for each installation.

Annual Routine Maintenance Costs

Annual routine maintenance costs, like surface preparation costs, are a function of pavement condition, local prices, and local installation policy. Each factor was determined using the same procedure as outlined for surface preparation cost. Total unit costs were calculated using Equation 2. Although the same density matrices were used, routine maintenance policy differed substantially from surface preparation policy, and thus the unit cost values for each distress type (C_{ij}) would be markedly different. An example of annual maintenance policy is shown in Table 4.

Pavement Performance

Life-cycle costing requires the determination of pavement service life and rate of performance deterioration. Therefore, a substantial effort was made in the development of PCI versus age relationship for each pavement class. The expected life of an M&R alternative is usually based on engineering judgment and experience, with consideration given to local materials, environmental factors, and traffic levels. However, this subjective evaluation usually leads to wide variation in estimated service life. Additionally, most definitions of service life and deterioration rates in the literature are usually not explicit and certainly not in terms of PCI values. In addition, as performance is so dependent on local materials and environmental factors, it would be difficult to relate service life for pavements from different locations. For this research project, it was decided to use the available data base to develop aggregate estimates of pavement performance.

To model pavement performance, both the graphical capabilities of the microcomputer data base manager, KMAN *(2)*, and the statistical procedures of the package SPSS *(3)* were used to test a large number of models. The best model was in the following form:

$$C = 100 - b\,x^m$$

where

C = pavement condition expressed in terms of PCI,

b = slope coefficient,

m = parameter whose value controls the degree of curvature of the performance curve, and

x = pavement age (months).

TABLE 1 DENSITY MATRIX FOR ALL CLASSES

Distress	Severity	Average	Density	(Percent)	By PCI Range	
Code	No.	81–100	61–80	41–60	21–40	0–20
1	1	0.32	1.15	5.54	11.36	9.46
1	2	0.13	0.20	1.80	10.24	14.09
1	3	0.03	0.04	0.39	1.05	14.02
2	1	0.30	0.89	1.04	1.02	1.25
2	2	0.02	0.17	0.37	1.08	0.56
2	3	0.00	0.00	0.00	0.01	0.01
3	1	0.57	11.45	6.90	5.46	3.23
3	2	0.06	0.93	5.80	8.01	7.42
3	3	0	0.01	0.13	0.59	2.58
4	1	0.01	0.01	0.03	0.02	0.01
4	2	0.00	0.00	0.00	0.01	0.05
4	3	0.00	0.00	0.00	0.00	0.02
5	1	0.01	0.17	0.15	0.03	0.07
5	2	0.00	0.00	0.01	0	0.19
5	3	0	0.00	0.00	0	0
6	1	0.05	0.11	0.25	0.26	0.28
6	2	0.01	0.03	0.15	0.17	0.47
6	3	0.00	0.00	0.03	0.05	0.16
7	1	0.53	0.66	0.72	0.70	0.48
7	2	0.22	0.88	1.51	1.26	1.10
7	3	0.04	0.20	0.50	0.81	2.25
8	1	0.13	0.23	0.23	0.03	0
8	2	0.06	0.25	0.30	0.18	0
8	3	0.01	0.02	0.03	0.02	0.01
9	1	0.27	0.15	0.13	0.11	0.13
9	2	0.31	0.44	0.22	0.22	0.10
9	3	0.14	0.26	0.34	0.21	0.05
10	1	1.53	2.83	2.50	1.57	0.86
10	2	0.30	0.82	1.41	1.78	1.05
10	3	0.01	0.03	0.08	0.16	0.16
11	1	0.32	0.74	1.19	1.67	0.79
11	2	0.09	0.20	0.84	1.15	2.44
11	3	0.00	0.02	0.14	0.44	1.13
12	1	0	0	0	0	0
12	2	0	0	0	0	0
12	3	0	0	0	0	0
13	1	0.00	0.00	0.01	0.03	0.28
13	2	0.00	0.00	0.01	0.02	0.19
13	3	0.00	0.00	0.01	0.02	0.37
14	1	0.10	0.02	0.01	0.11	0
14	2	0.01	0.02	0.00	0.06	0.09
14	3	0.04	0	0	0.08	0.36
15	1	0.11	0.36	1.17	1.80	2.39
15	2	0.04	0.10	0.73	0.80	1.66
15	3	0.07	0.05	0.18	0.45	2.02
16	1	0.01	0.05	0.01	0.01	
16	2	0.00	0.00	0.05	0.00	
16	3	0.00	0.00	0.00	0	0
17	1	0.00	0.00	0.01	0.04	0.00
17	2	0.00	0.00	0.02	0.00	0.00
17	3	0	0	0	0.00	0.02
18	1	0.00	0.00	0	0	0.00
18	2	0	0.00	0	0.00	0.01
18	3	0	0	0	0	0
19	1	4.49	12.44	17.15	19.57	11.25
19	2	0.56	1.18	6.60	10.84	17.31
19	3	0.06	0.08	0.42	6.04	23.74

TABLE 2 IDEAL SURFACE PREPARATION POLICY: FORT KNOX

Distress Type	Severity	Method	Unit	Unit Cost ($)
Alligator cracking	H	Deep patch	SF	2.98
	M	Shallow patch	SF	1.78
	L	Seal coat	SF	0.12
Bleeding	H	Seal coat	SF	0.12
	M	Seal coat	SF	0.12
Block cracking	H	Shallow patch	SF	1.70
	M	Seal coat	SF	0.12
Bumps/sags	H	Shallow patch	SF	1.78
	M	Skin patch	SF	1.01
Corrugation	H	Shallow patch	SF	1.78
	M	Skin patch	SF	1.01
Depressions	H	Shallow patch	SF	1.78
	M	Skin patch	SF	1.01
Edge cracking	H	Deep patch	LF	4.47
	M	Shallow patch	LF	2.23
Lane/shoulder dropoff	H	Grade and add gravel	LF	0.38
	M	Grade and add gravel	LF	0.28
Longitudinal transverse cracking	H	Crack seal	LF	1.42
	M	Crack seal	LF	1.01
	L	Crack seal	LF	0.31
Patching and utility cut patching	H	Replace patch	SF	2.98
	M	Crack seal	SF	1.68
Potholes	H	Deep patch	Each	9.36
	M	Deep patch	Each	2.32
	L	Shallow patch	Each	1.39
Rutting	H	Deep patch	SF	2.98
	M	Shallow patch	SF	1.78
	L	Skin patch	SF	1.01
Shoving	H	Shallow patch	SF	1.78
	M	Shallow patch	SF	1.78
Slippage crack	H	Shallow patch	SF	1.78
	M	Shallow patch	SF	1.78
Swell	H	Shallow patch	SF	1.78
	M	Shallow patch	SF	1.78
Weathering and raveling	H	Seal coat	SF	0.12
	M	Seal coat	SF	0.12
	L	Seal coat	SF	0.12

NOTE: H = high, M = medium, L = low; SF = square ft, LF = linear ft.

TABLE 3 SAMPLE CALCULATION OF SURFACE PREPARATION COSTS

Density	Distress Type	Severity	Preparation Method	Unit	Unit Cost ($)
0.55	Alligator cracking	Medium	Deep patch	SF	3.80
0.12	Alligator cracking	High	Deep patch	SF	3.80
1.25	Longitudinal/transverse cracking	High	Crack seal	LF	1.15

The surface preparation unit cost ($) can then be calculated as follows:

Medium alligator cracking	=	$0.55 \times 0.09 \times 3.80 =$	0.19 yd^2
High alligator cracking	=	$0.12 \times 0.09 \times 3.80 =$	0.04 yd^2
High longitudinal/transverse cracking	=	$1.25 \times 0.09 \times 1.15 =$	0.13 yd^2
Total surface preparation cost			$0.36 yd^2

TABLE 4 ANNUAL MAINTENANCE POLICY: FORT KNOX

Distress Type	Severity	Method	Unit	Unit Cost ($)
Alligator cracking	H	Deep patch	SF	2.98
	M	Skin patch	SF	1.01
Block cracking	H	Shallow patch	SF	1.70
Bumps/sags	H	Shallow patch	SF	1.78
Corrugation	H	Shallow patch	SF	1.78
Depressions	H	Shallow patch	SF	1.78
Edge cracking	H	Deep patch	LF	4.47
Lane/shoulder dropoff	H	Grade and add gravel	LF	0.38
Longitudinal/transverse cracking	H	Crack seal	LF	1.42
	M	Crack seal	LF	1.01
Patching and utility cut patching	H	Replace patch	SF	2.98
	M	Crack seal	SF	1.68
Potholes	H	Deep patch	Each	9.36
	M	Deep patch	Each	2.32
Rutting	H	Skin patch	SF	1.01
Shoving	H	Skin patch	SF	1.01
Slippage crack	H	Skin patch	SF	1.01
Swell	H	Skin patch	SF	1.01

NOTE: H = high, M = medium, L = low; SF = square ft, LF = linear ft.

TABLE 5 COST AND PERFORMANCE DATA FOR DIFFERENT M&R ALTERNATIVES ON THIN OVERLAY PAVEMENT: FORT EUSTIS

COST DATA

Initial Cost (Fixed) of Different M & R Alternatives

M & R Activity:	Surface Treatment	Thin Overlay	Thick Overlay	Recon-struction
Unit cost ($/sy):	1.58	3.76	5.07	20.70

Initial Cost (Surface Preparation at the Time of Repair)

PCI Range:	0-20	21-40	41-60	61-80	81-100
Unit Cost ($/sy):	18.50	8.50	5.20	0.51	0.15

Annual Maintenance Cost of Different M & R Activities

PCI Range:	0-20	21-40	41-60	61-80	81-100
Unit Cost ($/sy) for:					
- Surface Treatment	7.7	2.2	0.80	0.50	0.13
- Thin Overlay	7.0	2.0	0.70	0.35	0.13
- Thick Overlay	4.0	1.0	0.60	0.30	0.07
- Reconstruction	4.4	1.3	0.65	0.33	0.07

PERFORMANCE

Surface Treatment	$PCI = 100 - 0.0319\,(age)^{1.5}$
Thin Overlay	$PCI = 100 - 0.0158\,(age)^{1.5}$
Thick Overlay	$PCI = 100 - 0.0129\,(age)^{1.5}$
Reconstruction (new Asphalt Pavement)	$PCI = 100 - 0.0104\,(age)^{1.5}$

The SPSS software *(3)* was used to develop regression equations for each pavement class. The following five variations of the general form of the performance equation were analyzed:

$PCI = 100 - b * Age^{1.5}$
$PCI = 100 - b * Age^{2.0}$
$PCI = 100 - b * Age^{2.5}$
$PCI = 100 - b * Age^{3.0}$
$PCI = 100 - b * Age^{4.0}$

The best fit was determined by the highest r^2 value (coefficient of determination) using the least-squares method. For all pavement classes at all installations, an exponent (m) of 1.5 resulted in the highest r^2 values. In this study, pavements were considered to have reached the end of their service life at the PCI level of 70. This value was chosen as the existing data base indicated that most installations were performing some form of repair activity on a pavement once it dropped below that level. In some instances, there were insufficient data samples to generate performance curves for all pavement classes. For pavements lacking regression equations, the general form of the equation was used with an exponent of 1.5. Next, the pavement service life, or age to PCI 70, was estimated. The regression equation's slope coefficient (b) could then be back calculated. Performance curves, regression equations, and r^2 values for each pavement class at all installations were calculated. The procedure to generate performance curves has now been automated *(4)*.

LIFE-CYCLE COST ANALYSIS

An economic cost comparison among M&R alternatives was performed by determining the overall life-cycle cost of each alternative. Life-cycle costs can be expressed as a present worth or equivalent uniform annual cost. If alternatives are to be compared using the present worth method, all alternatives must be evaluated over the same analysis period. If an alternative's service life exceeded the analysis period, then the worth of that remaining life (salvage value) has to be determined. The equivalent uniform annual cost method (EUAC) allows the comparison of alternatives over different analysis periods. The EUAC method combines all investment costs and all annual expenses into a single annual sum that is equivalent to all disbursements during the pavement's service life, if spread uniformly over that period. When alternatives are compared, the one with the lowest equivalent uniform annual cost is considered the most economical.

The procedure used for determining the equivalent uniform annual cost of different M&R activities is best illustrated through the use of an example. In Table 5, an example problem is presented along with the necessary cost and performance data. The selection of the best alternative procedure is presented as follows in a step-by-step format.

Step 1

Determine total initial cost of each M&R alternative as the sum of initial fixed cost and surface preparation cost. Surface preparation cost is a function of the PCI value at the time of repair and the installation surface preparation policy. For example, the total initial cost for surface treatment is equal to \$1.58 (fixed cost) + \$0.51 (surface preparation) = \$2.09/yd². Similarly, the total initial cost for a thin overlay, structural overlay, and reconstruction are \$4.27, \$5.58, and \$20.70/yd², respectively.

Step 2

Determine service life (number of years to reach a PCI value of 70) for each M&R alternative. Using the performance models given in Table 5, and solving for age at PCI = 70, the required service life is determined. For instance, in the case of surface treatment a period of approximately 96 months or 8 yr is required to reach a PCI of 70. Similarly, service lives for thin overlay, thick overlay, and reconstruction are 13, 15, and 17 yr, respectively.

Step 3

Determine Equivalent Uniform Annual Cost (EUAC) of initial cost of each maintenance alternative as follows:

$$EUAC = IC * (CRF, i, n)$$

where

IC = initial cost as determined in Step 1,

CRF = capital recovery factor = $\dfrac{i(1 + i)^n}{(1 + i)^n - 1}$,

i = inflation-adjusted discount rate (6 percent), and

n = service life as determined in Step 2.

Thus, the EUAC of initial cost of different maintenance alternatives is

Surface treatment = 2.09 (0.1610) = \$0.34/yd²
1.5-in. overlay = 4.27 (0.1130) = \$0.48/yd²
2.0-in. overlay = 5.58 (0.1030) = \$0.57/yd²
Reconstruction = 20.70 (0.0954) = \$1.97/yd²

Step 4

Determine the EUAC of annual maintenance through the service life of each M&R alternative. This is done by taking the following steps.

(*a*) Determine the PCI value at each year of the service life of an alternative. For example, it is required to know the 8 PCI values corresponding to each of the 8 years of the surface treatment service life. These values are obtained by using the performance models shown in Table 4. Using the performance model of surface treatment results in a PCI value of 93 at the third year of the service life (age = 36 months) and a PCI value of 75 at the 7th year (age = 85 months).

(*b*) For each year's PCI, as calculated in Step 4(*a*) determine the corresponding PCI range and the corresponding annual maintenance cost. For example, in the case of surface treatments, at the third year the PCI value is 93 and the corresponding PCI range is 81 to 100. Thus, the annual maintenance cost is \$0.13/yd², as indicated in Table 4.

Similarly, at the 7th year, PCI value is 75 and the PCI range is 61 to 80 and the associated annual maintenance cost is $0.50/yd².

(*c*) Determine the present worth value (PWV) of all annual maintenance costs determined in Step 4(*b*) as follows:

$$\mathrm{PWV} = \sum_{j=1}^{n} \mathrm{AMC}_j * (\mathrm{SPPWF},\ i,\ j)$$

where

PWV = present worth value of all annual maintenance costs during the service life of an alternative,
AMC_j = annual maintenance cost at the jth year of the alternative's service life,
$(\mathrm{SPPWF},\ i,\ j)$ = $\dfrac{1}{(1+i)^j}$ equals single payment present worth factor,
i = inflation-adjusted discount rate (6 percent), and
n = service life (yr) of the alternative under consideration, as determined in Step 2.

(*d*) Convert the PWV obtained in Step 4(*c*) to its EUAC as follows:

$$\mathrm{EUAC} = \mathrm{PWV} * (\mathrm{CRF},\ i,\ n)$$

where

EUAC = equivalent uniform annual cost ($/yd²/yr) of the maintenance alternative under consideration,
PWV = present worth value as defined in Step 4(*c*), and
$(\mathrm{CRF},\ i,\ n)$ = capital recovery factor, as defined in Step 3.

Executing calculations in Steps 4(*a*) through 4(*d*) for different maintenance alternatives results in EUAC of annual maintenance of $0.21, $0.18, $0.11, and $0.13/yd² for surface treatment, thin overlay, structural overlay, and reconstruction, respectively.

Step 5

Determine the total EUAC of each alternative by adding values from Steps 3 and 4.

EUAC (Surface treatment) = $0.34 + $0.21 = $0.55/yd²
EUAC (Thin overlay) = $0.48 + $0.18 = $0.66/yd²
EUAC (Structural overlay) = $0.57 + $0.11 = $0.68/yd²
EUAC (Reconstruction) = $1.97 + $0.13 = $2.10/yd²

Step 6

Select the repair alternative with the least equivalent uniform annual cost.

The life-cycle cost analysis of the example problem has shown that alternative No. 1 (surface treatment) has the least equivalent uniform annual cost. This alternative would cost Fort Eustis the equivalent of a yearly payment of $0.55/yd² over an 8-yr period at the assumed interest and inflation rates. It should be noted that the user costs associated with pavement conditions and lane closures were not included in the analysis, but would probably not affect the results much as traffic levels are relatively light.

The procedure presented above was repeated for different PCI ranges and the results are summarized in Figure 1. For

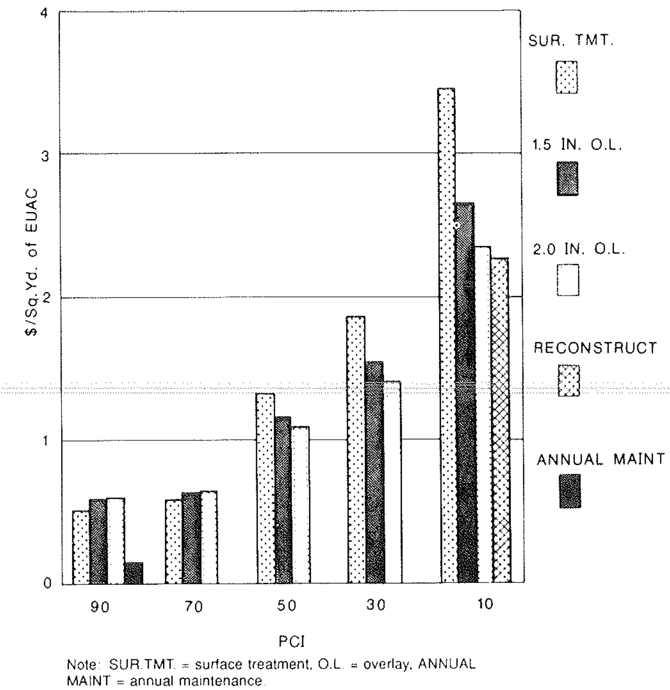

FIGURE 1 Equivalent uniform annual costs of different M&R alternatives for thin overlay pavement by PCI range: Fort Eustis.

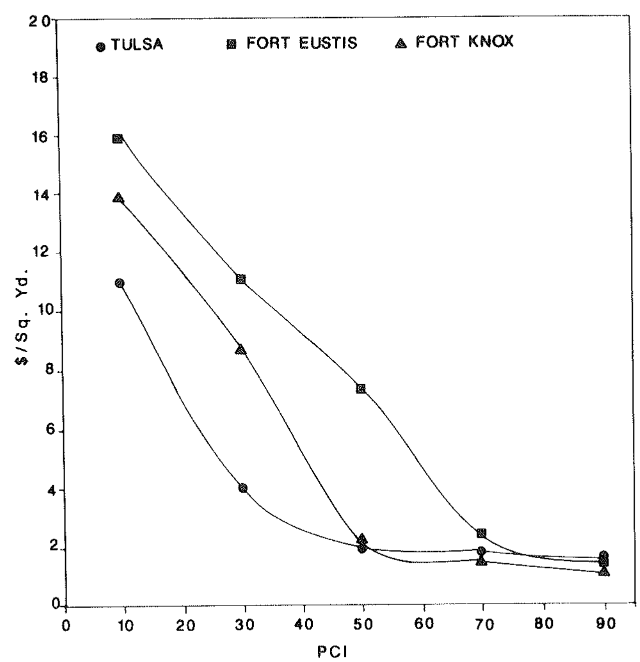

FIGURE 2 Initial costs of least-cost M&R alternatives for asphalt concrete roads.

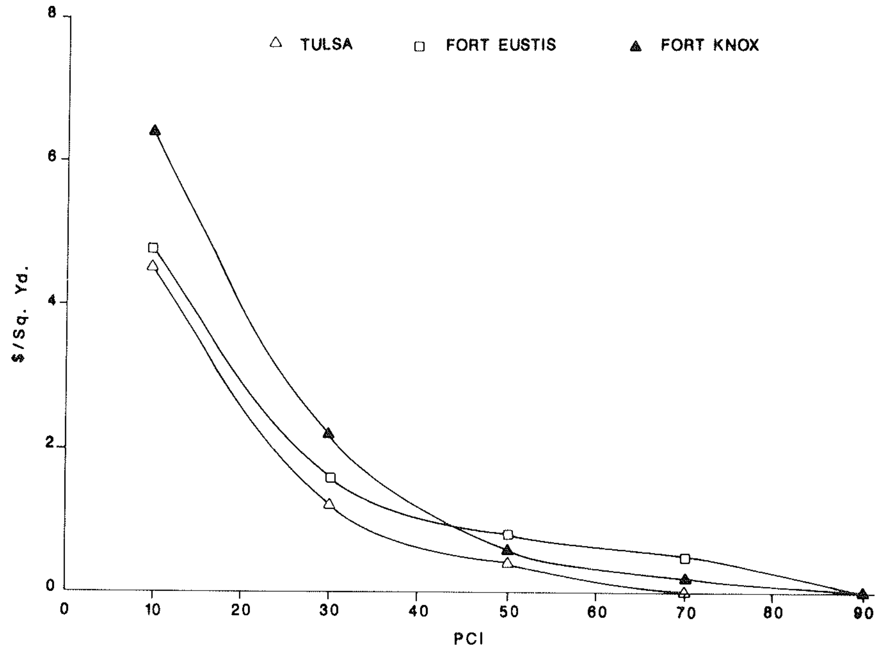

FIGURE 3 Annual routine maintenance costs of least-cost M&R alternatives for asphalt concrete roads.

TABLE 6 RESULTS OF SENSITIVITY ANALYSIS

Analysis Parameter	Percent Change in EUAC due to One Percent Change in Analysis Parameter			
	Surface Treatment	Thin Overlay	Thick Overlay	Recon-struction
Service Life	0.50	0.60	0.60	0.60
Discount Rate	0.20	0.25	0.40	0.40
Initial Cost	0.60	0.80	0.90	1.00
Annual Maint. Cost	0.40	0.25	0.16	0.04

instance, although surface treatment is the most cost-effective maintenance alternative at the PCI range of 61 to 80, structural overlay is the most cost effective at the PCI range of 41 to 60 and reconstruction is the best alternative at the PCI range of 0 to 20. Similar computations were done for all pavement classes and for all PCI ranges for each of the five installations, and the best economic repair alternatives under various conditions were determined. Figures 2 and 3 present the initial costs and annual routine maintenance costs, respectively, associated with the least cost alternatives at various PCI ranges for three installations.

Sensitivity Analysis

The results of the above example are only as good as the estimates of service life, initial cost, annual maintenance expenses, and effective discount rate used. A sensitivity analysis was included to gauge what effect each of these estimates would have on life-cycle costs. Estimates were made with different values for each of the parameters associated with various alternatives. The effect is presented in Table 6 as the percent change in EUAC due to 1 percent change in an analysis parameter. For example, 1 percent change or error in estimating the service life of a surface treatment results in, on the average, 0.5 percent change in the overall EUAC. Similarly, 1 percent difference in the initial cost of reconstruction results in, on the average, 1 percent difference in the overall EUAC.

The results of this sensitivity analysis indicate that the accuracy of the calculated equivalent uniform annual costs of M&R alternatives is very sensitive to errors in input initial cost and expected service life. Incorrect estimation of annual maintenance expenses would not greatly affect the final EUAC values. Also, variations in discount rates did not seem to be as critical as a miscalculation of initial cost or service life.

CONCLUSION

The paper presented a methodology for determining the least-cost maintenance and repair alternative for different pavement categories at various PCI ranges. The data from five military

installations from across the United States were used. Although the case study results suggest that the methodology is reasonable, further work is necessary with an expanded data base from geographically representative military installations.

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New Techniques for Modeling Pavement Deterioration

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The ability to model and predict pavement condition accurately is critical to the success of pavement management systems. This paper evaluates the applicability of three mathematical curve-fitting techniques for modeling pavement condition deterioration behavior. The mathematical models investigated are: stepwise regression, *B*-spline approximation, and constrained least-squares estimation. The best features of each are integrated into an interactive format capable of operating within the PAVER pavement management system. Pavement sections from a given location consisting of the same pavement type, use, and rank are grouped into families. Models that filter obvious errors and statistical outliers from the data are applied to the family data. Both the *B*-spline approximation and constrained least-squares techniques are used in the screening process. A constrained least-squares curve-fitting technique is used to fit a family prediction model curve to the filtered data. Pavement condition prediction at the section level is accomplished using the position of a section relative to the family prediction model curve. Extrapolation of pavement family condition values beyond the maximum age available in a family data base is accomplished using a backtracking method. These methods were determined to give the most consistent and believable results from among a number of possible methods considered. These procedures constitute a complete method to model and predict pavement family behavior and pavement section behavior accurately. The method is being integrated into the PAVER pavement management system to improve the pavement evaluation process.

The ability to predict pavement deterioration accurately is critical to the success of any pavement management system. A successful pavement condition prediction technique provides a fundamental tool to aid in the planning and cost allocation of maintenance and rehabilitation activities. Detailed in this paper are the results of a study undertaken to develop a reliable pavement condition modeling technique.

The prediction methodology presented herein was developed for incorporation into the PAVER pavement management system (PMS) *(1)*. However, other management systems that use historical condition data can also effectively use this prediction technique. PAVER was designed to optimize fund allocation for pavement maintenance and repair. Currently, PAVER employs a straight-line extrapolation applied to each individual pavement section based on the results of previous condition surveys to model pavement condition. However, when pavement condition index (PCI) versus age is plotted, a curvilinear relationship with monotonically decreasing PCI is expected. Thus, a need clearly exists for a more realistic pavement condition modeling technique. The methodology presented in this paper is based on several years of research to improve the current pavement condition modeling and prediction procedures in PAVER.

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SUMMARY OF PREVIOUS RESEARCH

Most PAVER data bases currently contain information pertaining to a one-time survey for each section during the lifetime of the pavement. Therefore, a critical need exists for a model to predict pavement behavior when good historical condition data for a section are unavailable. Several studies have been undertaken in the past to satisfy the need for an accurate method of pavement condition prediction.

Initially, stepwise regression models were used to predict pavement condition index (PCI). Several variables were incorporated into these models, including pavement type, condition rating, nondestructive testing (NDT) information, pavement construction, traffic information, pavement age, and pavement layer thicknesses. These models were evaluated and determined unsatisfactory for predicting PCI at the project level *(2)*. The resultant R^2 was very low and the residual standard error was high when stepwise regression was used to model these data. This was partially attributed to the large number of estimating errors associated with the models. Another drawback to these complex models is that they are difficult to update to reflect additional condition data as they are collected.

In an attempt to address these problems, Nunez and Shahin *(3)* took another approach to modeling pavement behavior. To combat the problem of insufficient section-specific condition history data, a family approach was developed. Pavement sections from a given PAVER data base that had the same pavement type, pavement use, and pavement rank were grouped into families. In this approach, sections with different ages and condition ratings are assumed to represent the deterioration in condition of a typical family section over time. Thus, by collecting each section age and condition, placing them chronologically, and fitting a curve through the points, a good idea of the total performance over time that is expected for all of the family sections can be obtained.

A family is defined by any combination of the following: *(a)* the pavement type: asphalt concrete (AC), portland cement concrete (PCC), asphalt concrete overlay on portland cement

concrete (APC), and asphalt concrete overlay on asphalt concrete (AAC); *(b)* the pavement use: identified by the service rendered, such as roadways, streets, parking lots, runways, taxiways, or aprons; and *(c)* the pavement rank or functional classification, such as arterials, collectors, and local roads and streets. The criteria for pavement family selection are as follows:

Surface type

AC	=	asphalt concrete
PCC	=	portland cement concrete
AAC	=	asphalt concrete overlay on asphalt concrete
APC	=	asphalt concrete overlay on portland cement concrete

Branch use

MT	=	motorpool
ST	=	storage
RO	=	roadway
PA	=	parking
RV	=	runway
AP	=	apron
HE	=	helipad
TA	=	taxiway

Pavement rank

A	=	principal
B	=	arterial
C	=	collector
D	=	industrial
E	=	residential

The family models were designed to allow continual updating as more data are gathered for a given location and entered in the data base.

This grouping of pavement sections with similar characteristics into families is the equivalent of including three additional variables in the model: pavement type, traffic, and pavement use. Furthermore, by grouping together pavements from individual geographic locations, climate is implicitly included as a variable in the family models. This family grouping makes it possible to develop reasonably accurate relationships for pavement condition prediction. Whereas other variables besides pavement type, traffic, pavement use, age, and climate certainly play a role in determining pavement condition, the availability of any such additional data in the PAVER data bases is currently limited or nonexistent. The present definition of family, which was used in the study by Nunez and Shahin *(3)* and also in this study, is provisional and will be further refined as more data become available. Efforts are not being made to incorporate a cause-of-distress factor, structural versus nonstructural, into the family definition *(4)*.

A screening procedure was designed *(3)* to examine the retrieved family data for obvious errors, and a statistical outliers analysis was implemented to detect any unusual observations. After the family data had been screened for these errors and outliers, polynomial curves were then fitted to the PCI versus age data points. It was found that a third-order polynomial curve best modeled the relationship of PCI versus age. This polynomial technique proved inadequate because a curvilinear relationship with monotonically decreasing PCI is expected when PCI versus age is plotted. The resultant third-order polynomial curve, when fitted to a family's data points, did not universally follow this known trend as the curve was not always smooth or monotonically decreasing.

Nunez and Shahin attempted to solve this problem by grouping data points within a family into 3-year averages and a third-order polynomial curve was fitted through these representative points. This was done under the assumption that the average pavement condition for the 3-year age ranges will influence the regression curve equally, regardless of the number of points in every range. However, the resultant curve was still not always smooth or monotonically decreasing. Therefore, the search for an applicable mathematical modeling scheme that would ensure decreasing PCI values as the age variable increases was expanded to include new methodologies.

APPROACH FOR NEW METHODOLOGY

Several requirements for the suitability of a particular method for pavement condition modeling and prediction must be met, including:

1. The PCI cannot be greater than 100 or less than zero,
2. The function must have a PCI equal to 100 at age equal to zero,
3. The function representing PCI versus age must be strictly decreasing as the age value increases,
4. The procedure must be suitable for automatic processing,
5. The procedure must be fast and capable of handling several hundred observations,
6. The procedure must be capable of being updated to reflect the addition of new condition survey data to the data base,
7. The procedure must be capable of being integrated into the current PAVER system without being obtrusive to the user, and
8. The procedure must accept user input information in developing family curves.

To develop an acceptable model for PCI prediction, several mathematical curve-fitting techniques were applied to historical pavement condition data. First the data were grouped into families and then subjected to a filtering procedure and an outliers analysis, as in the previous study by Nunez and Shahin *(3)*. Two polynomial curve-fitting techniques based on B-spline approximation and constrained least-squares estimation were applied to the filtered family data to determine their ability to satisfy the previously defined requirements. Extrapolation techniques that allow prediction of PCI values beyond the last PCI versus age data points were also studied. The following sections present the results of these investigations, including a summary of the drawbacks and advantages of using these curve-fitting and extrapolation techniques for the purpose of predicting future pavement condition.

PAVEMENT CONDITION DATA

PAVER Data Bases

To establish a modeling procedure that is representative and applicable throughout the United States, a comprehensive selection of pavements must be evaluated using that procedure.

The data bases used in this study came from cities, counties, and military installations that have adopted PAVER. Data in the PAVER system are stored on a section-by-section basis. Data normally available for each section include: pavement section identification, functional classification, age since construction or major rehabilitation, PCI, layer-material properties and, in some cases, traffic records. Table 1 summarizes the location of the data bases used in this study.

TABLE 1 SUMMARY OF THE NUMBER OF CASES BY DATA BASE

Data Base	No. of Cases
Abilene, Tex.	214
Ada, Idaho	742
Bellingham, Wash.	263
Billings, Mont.	93
Bloomingdale, Ill.	163
Bryan, Tex.	336
Calgary, Alta., Canada	891
Glenn Ellyn, Ill.	271
Hayward, Calif.	1,236
Niagara, Ont., Canada	617
Overland Park, Kans.	305
Tacoma, Wash.	314
Winnipeg, Man., Canada	113
Fort Eustis, Va.	221
Great Lakes, Ill.	559
Total	6,338

The PCI of a pavement is determined from the type, severity, and quantity of distress and is a composite index of the pavement's structural integrity and operating condition. The PCI is expressed as a numerical value ranging from 0 to 100 that has been divided into seven descriptive categories, ranging from failed to excellent. Based on the value of the PCI, an appropriate level of maintenance and rehabilitation can be recommended.

Family Definition: Data Retrieval and Organization

Data retrieval is accomplished with an automatic extraction program that selects information about the pavement section identification, PCI, and age. This information is retrieved based on the user-specified definition of a pavement family. A pavement family is defined as a group of pavement sections with the same pavement type, pavement use, and pavement rank. The ability of the users to set family definitions that may be unique for their particular location provides freedom to develop models specifically for that particular location.

Data Screening

Data Filtering Procedure

After data are retrieved it is necessary to filter out the inaccurate data. This is accomplished through a specially developed computer program. In the data-filtering procedure, the family data are first sorted by pavement section identification number, age, and PCI. When the same section is listed more than once, sequential cases of the same section are compared. If the PCI increases with age and the increase is greater than 20 points, the case with the higher PCI is removed to the errors file. This condition indicates that either an error is present in one of the records or that major rehabilitation has been performed between condition surveys, which would place this section in a different family of pavements. If a pavement section of the same age is listed more than once and the PCIs are the same, only one pavement section is retained. If the PCIs are different for the same section and age, all cases are removed to the errors file.

A further check on spurious data is done using a set of PCI boundaries *(3)*. These boundaries are defined by the maximum and minimum PCI values expected over the life of the pavements. A user can set these boundaries or use the default values, which are based on observations of available data bases combined with engineering judgment. If a record falls outside either the defined upper or lower boundary, the record is removed to the errors file. The user can then examine this file and check these data for possible problems. An adjustment to the PCI envelope can then be made to incorporate the data points that were removed to the errors file if, after examination, the user believes these data to be accurate. An example output from the data-filtering procedure is shown in Figure 1.

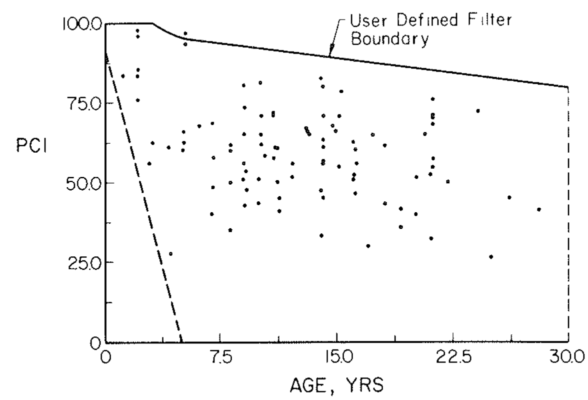

FIGURE 1 Example output of data-filtering procedure.

Outlier Analysis Procedure

The data-filtering procedure removes obvious errors from family data. To ensure appropriate model building, further examination of the data for removal of extreme observations is performed in the outliers analysis procedure. This step is important because cases with unusual performance can have a substantial impact on the statistical modeling of family behavior. The outlier procedure is performed by examining the residuals from PCI versus age curve fitting. In the previous study *(3)*, the residuals were calculated as the difference between the observed value and the value predicted by a linear regression model of PCI against age. In the current procedure, a constrained least-squares technique replaces the linear regression technique because the shape of the constrained least-squares model more closely resembles accepted pavement deterioration behavior.

A program was written to generate the residuals that are calculated as the difference between the observed PCI values

and the predicted PCI values. The frequency distribution of residuals was found to be normal *(3)*, which allowed setting confidence intervals for scaling the spread of the data. The outlier program was developed to allow for various confidence limits (i.e., 95 percent), to be established by the user for removal of extreme cases. Figure 2 is an example output from the outlier analysis procedure.

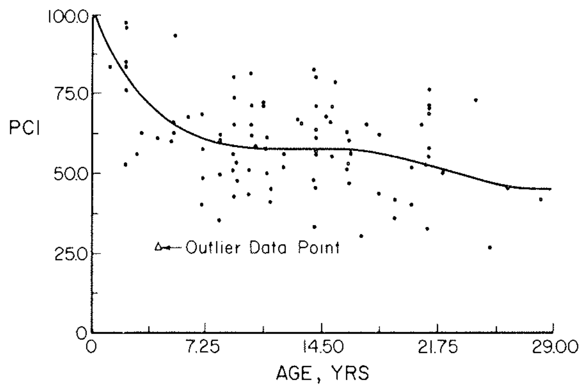

FIGURE 2 Example output of outlier analysis procedure.

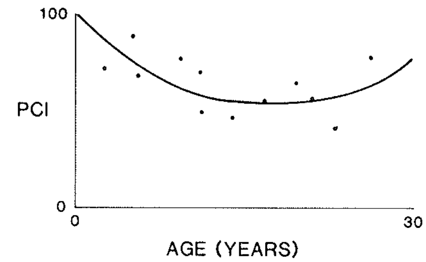

FIGURE 3 *B*-spline approximation: annual data averaging.

DEVELOPMENT OF MATHEMATICAL MODELS

B-spline Approximation

To address the problems resulting from using a polynomial curve fitting procedure, a normalized *B*-spline function *(5)* was chosen to approximate the PCI/age data. Spline functions can best be explained as mechanical splines, which are flexible strips of elastic material. The mechanical spline is secured by means of weights at selected points, referred to as knots, through which the draftsman wishes the spline to pass. The spline assumes a shape that minimizes its potential energy. Elementary beam theory suggests that the function describing this mechanical spline is a cubic polynomial between each pair of knots. Adjacent polynomials join continuously with continuous first and second derivatives.

B stands for basis, or for the bell shape that characterizes such functions. A *B*-spline of degree k is a continuous function with its first $(k - 1)$ derivatives being continuous. In approximating several PCI/age families, it was found that *B*-splines of degree as low as 3 are sufficiently smooth to be useful.

An important consideration in using this mathematical model is the choice of knots for these *B*-splines. Sensible choices of the number and positions of the interior knots may often be estimated by examining the shape of the desired curve, but generally this is a process that requires advanced engineering judgment. Failures in this selection process can result in functions that are not strictly decreasing.

Initially, age data were averaged on an annual basis. This still produced an occasional positive trend in the function, as shown in Figure 3, a condition defined previously as unacceptable. In an attempt to rectify this situation, the age data were averaged on a 3- and 5-year basis. This produced smoother curves and a reduced occurrence of positive slopes, although functions that are not strictly decreasing can still result.

There are several causes for the *B*-spline function to take on a positive slope. One is the presence of more than one family in the retrieved data, as illustrated by Figure 4. Another possibility for an increasing trend in the *B*-spline function is a case in which a section is shown to have an unexpectedly high PCI at some age. The *B*-spline function will follow the data trend regardless of error, as shown in Figure 5. Note that when the extreme data point is removed, the *B*-spline function no longer exhibits a positive slope. Finally, a poor choice of the number and positions of the knots can result in functions that are not strictly decreasing.

Because of the complex nature of selecting the interior knots and the possibility of the occurrence of a positive trend in the function, the *B*-spline technique was not deemed suitable for inclusion in the pavement prediction modeling technique for PAVER. However, it can be used effectively in analyzing data and warning the user of potential problems and assisting him in identifying them. These uses of the *B*-spline approximation model will be discussed further in a later section. Another

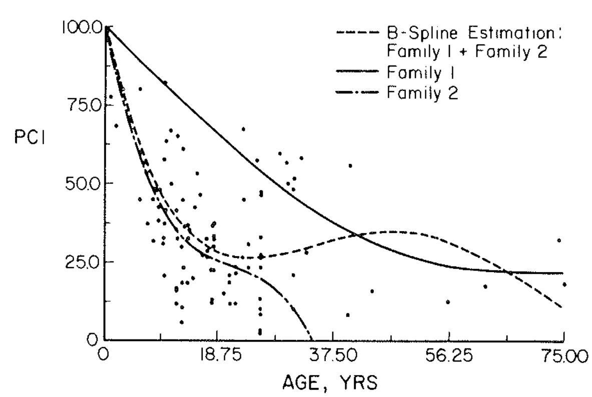

FIGURE 4 *B*-spline approximation: data file containing two families.

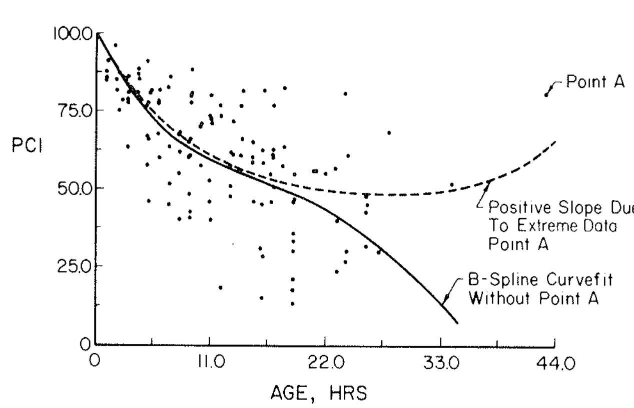

FIGURE 5 *B*-spline approximation: data file containing extreme data point.

approximation scheme, the constrained least squares, was adopted for integration into the PAVER prediction technique.

Constrained Least Squares

Constrained least-squares estimation *(6)* approximates the given observations using polynomial curve fitting. Data analysis showed no reason to use a polynomial of degree exceeding 4 or 5. The curves are mathematically constrained by the requirement that the first derivative of the curve at any age is kept negative or zero. This ensures that PCI values do not increase with age. The following constrained linear least-squares function was adopted:

$$\text{Minimize} \quad \sum_{k=1}^{N} [y_k - P(x_k)]^2$$

Subject to $P(0) = 100$ (initial PCI)
$P'(x_j) < 0$ for any x_j between 0 and maximum age.

Here, $P(x)$ denotes a polynomial value of degree υ at age x, i.e., $P(x) = \alpha_0 + \alpha_1 x + \alpha_2 x^2 + \ldots + \alpha_\upsilon x^\upsilon$.

Note that $\alpha_0 = P(0) = 100$.

Using constrained least-squares estimation to model the relationship between PCI and age of a pavement has several advantages. Whereas *B*-spline approximation may result in the best fit of data points when all data points are accurate and belong to a single family, constrained least squares avoids the potential problems that a *B*-spline function may present. Constrained least-squares curves, unlike *B*-spline curves, will never exhibit a positive slope; that is, the PCI values are not allowed to increase with age. Also, constrained least-squares estimation is simpler to use and, if the data file contains errors or data from more than one family, it is more accurate than the *B*-spline approximation.

In summary, the constrained least-squares estimation model can obtain reasonable accuracy without incurring the problems associated with *B*-spline approximations. Shown in Figure 6 are the results of fitting an accurate, single-family data file with *B*-spline and constrained least squares. It can be seen that the curves are reasonably similar. It is advantageous to use the constrained least-squares estimation on a file that contains data errors or mixed family data. The *B*-spline curve may reveal a positive trend in these cases, whereas the constrained least-squares curve will not. Thus, constrained least squares will more accurately predict normal pavement deterioration behavior.

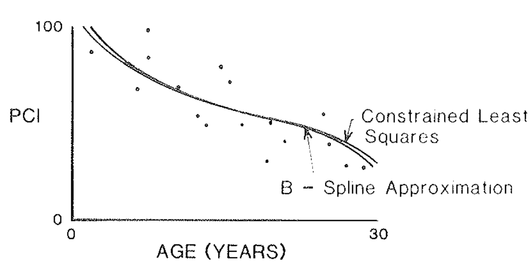

FIGURE 6 ***B*-spline versus constrained least-squares curve-fitting techniques: accurate data file.**

APPLICATIONS

Data Screening

Figure 7 contains the flow diagram depicting the iterative procedure a PAVER user will follow when defining pavement families and modeling their condition deterioration. First, the user defines the pavement family desired, under "Family Definition". Once all the data for that family have been retrieved, the data are filtered to eliminate inaccurate data in "Data Filtering for Errors". Next, the pavement family is subjected to an analysis of residuals based on constrained least squares "Outlier Analysis".

After all data have been screened in the data filtering procedure and outliers-analysis procedure, *B*-spline approximation is applied to the data to check PCI versus age slope. It was shown previously in this paper that a *B*-spline curve may take on a positive slope if errors are present in the data or if there is a mix of more than one family's data in the file. Although this ability to take on increasing PCI values as age increases limits the usefulness of *B*-spline approximation for prediction, it makes it very useful for identifying inconsistencies in the family's data file. If a positive slope does occur, the user has the option of returning to "Data Filtering", returning to "Family Definition", or continuing on to the portion of the program that applies a constrained least-squares curve-fitting technique to model the data.

Pavement Condition Extrapolation

The ability to predict pavement condition beyond the maximum age available in the data base is needed for PCI prediction into the future for the purpose of budget optimization.

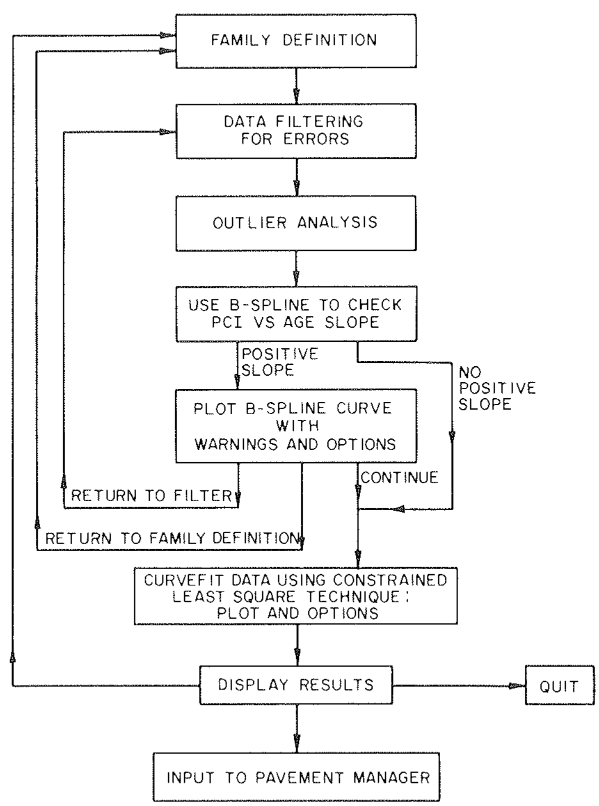

FIGURE 7 Flow diagram of family definition and curve-fitting procedure.

Both family condition and section condition must be extended beyond the available data. First, the family curve is extended beyond the age contained in the family data base. Next, pavement section condition must be predicted in a two-part procedure: *(a)* a curve must be established for a particular pavement section, and *(b)* this section curve must be extrapolated beyond the data points for that particular pavement section. This two-part section condition extrapolation is necessary to provide future predictions that provide crucial inputs into the maintenance and rehabilitation planning and economic analysis routines of PAVER. Both family and pavement section condition projection are discussed in the following sections.

Pavement Family Condition Extrapolation

The first approach taken to extend family prediction curves beyond the available data was based on using a polynomial developed from the constrained least-squares estimation. In order to use this constrained least-squares scheme for prediction beyond the maximum age in the observation table, the constraints were extended so that they would hold for any age between zero and the maximum age plus any desired number of years (e.g., 20). However, this approach produced curves with very steep slopes at the longer age values. An unacceptably rapid failure rate for the pavements resulted.

Applications of different straight-line extrapolations beyond the last point in the data base were also studied. Extension with a line of the same slope as that of the tangent to the curve at the last data point had the disadvantage of being unpredictable. The approach adopted was to use a backtracking method. The slope of the line between the last data point and the data point corresponding to an age 3 years before the last data point is determined. This slope is used to extend the curve beyond the last data point. This method, as shown in Figure 8, was determined to give the most consistent and believable results from among a number of possible methods considered.

Pavement Section Condition Extrapolation

The method used to extrapolate section condition is essentially the same as that proposed previously *(3)*. At the project level,

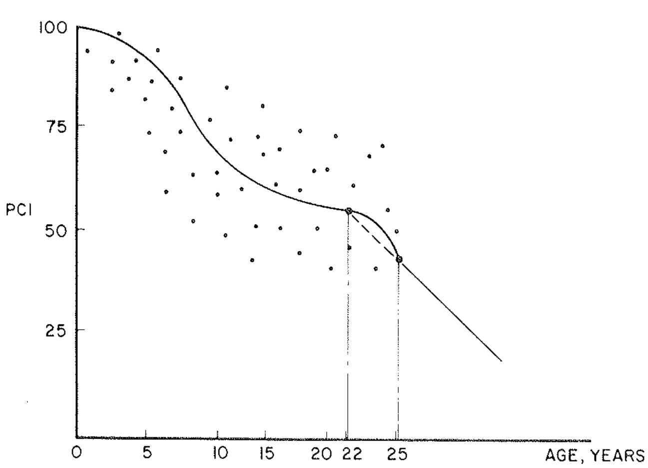

FIGURE 8 Pavement family condition extrapolation.

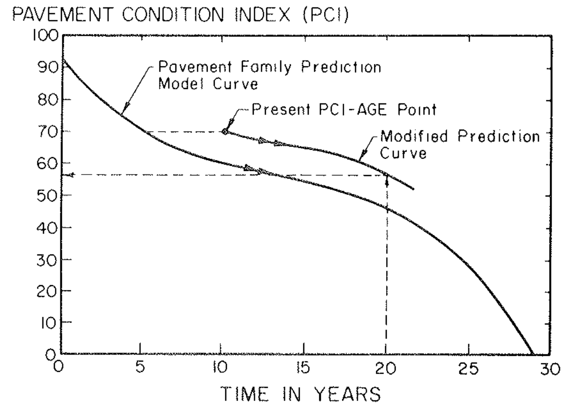

FIGURE 9 Pavement section condition extrapolation.

condition predictions are used to develop maintenance and repair alternatives. PCI prediction at the section level uses the pavement family prediction model curve. The prediction function for a pavement family represents the average behavior of all the sections of that family. Because climate, traffic, pavement type, and pavement use are the same for all sections in a pavement family, using the family curve should provide a reasonable estimate of section behavior.

The prediction for each section is done by using its relative position to the family prediction curve. It is assumed that the deterioration of all pavements in a family is similar and is a function of their present condition only regardless of age. A curve is drawn through the PCI-age points for the pavement section being investigated, parallel to the family prediction curve as shown in Figure 9. Mathematically, this is done by finding the roots of the polynomial function corresponding to the PCI value for the section. The PCI can then be determined at the desired future age.

SUMMARY

Pavement management relies heavily on the ability to predict pavement condition in the future based on historical data. By grouping pavement sections with the same pavement type, use, and rank together it was possible to develop reasonably accurate relationships for predicting pavement condition. The technique presented for modeling pavement deterioration behavior was developed specifically for use in PAVER, but it may be effectively used by other pavement management systems that use historical condition data.

After pavement sections have been drawn from a data base and grouped into families, the data pertaining to these sections are screened. A data-filtering procedure removes obvious errors and an outlier analysis procedure removes any exceptional pavements based on examination of the residuals of a constrained least-squares estimation model.

Once the initial screening has been accomplished, a *B*-spline approximation is applied to the data. If a positive slope results in the *B*-spline curve, it is taken as a warning. The data file is examined to determine whether more than one family of pavements is represented in the data file or if the presence of data errors is causing the increasing trend in the curve. The user then has the option of continuing the procedure or returning to the family definition or data-filtering phases of the program.

The technique selected for incorporation into PAVER to model the relationship between PCI and age of a pavement was constrained least-squares estimation. The procedure includes a least-squares estimation scheme that approximates the given observations with the constraint that the slope of the curve at any age must be kept negative or zero.

It is necessary to extrapolate pavement deterioration behavior beyond the maximum age available in a data base. Extrapolation of pavement family condition values is accomplished by using a backtracking method. PCI prediction at the section level is accomplished using the position of a section relative to the family prediction curve.

These procedures present a complete method to model and predict pavement family behavior and pavement section behavior. The present definition of a family is provisional and will be developed further as the data become more refined. This modeling technique was designed so that when more data are incorporated into the data base the model does not become obsolete but rather is improved by the increment in the number of observations. The modeling is simple, fast, and reliable. The model is being integrated into the PAVER program to improve the pavement evaluation process.

ACKNOWLEDGMENT

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Pavement Management at the Local Government Level

C. L. MONISMITH, F. N. FINN, J. A. EPPS, AND M. KERMIT

Described in this paper are the results of a study of pavement management practices used by 13 local government agencies in the United States and Canada. Included is a discussion of factors to be considered in planning and developing a pavement management system based on the experiences of these organizations. In the planning phase consideration is given to resource requirements (personnel, equipment, and funds) and to information requirements (primarily the type of data to be collected). Specific considerations associated with actual development include: *(a)* section identification, *(b)* condition surveys, *(c)* maintenance and rehabilitation alternatives, *(d)* data utilization, and *(e)* report preparation. Practices of the 13 agencies relative to these considerations are summarized in a series of tables for ready reference. Development and annual costs as well as personnel requirements of the 13 systems are summarized. Development costs of the order of $100 to $300/mi appear to represent a reasonable range that might be anticipated. Annual costs to operate the system of about $100/mi are considered average. While personnel requirements will vary depending on the size of the system, it is important to recognize that one engineer within the organization should be responsible for and fully knowledgeable of the system.

At every level of government, insufficient funds are available to maintain our street and highway systems at current levels of serviceability *(1)*. Accordingly, public funds that have been earmarked for pavements must be used as effectively as possible. A proven way to mitigate the effects of these funding problems is through the use of pavement management considerations *(2, 3)*.

Considerable effort is now under way at the state level to implement working pavement management systems, and a number of states are already effectively using pavement management techniques for maintenance and rehabilitation activities, e.g., the states of Arizona *(4)*, California *(5)*, and Washington *(6)*.

At the local government level efforts are also under way to implement pavement management systems. Described by Monismith et al. *(7)* are a survey and evaluation of a number of these agencies that have already initiated such activities. [An overall indication of the state of pavement management activities as of 1985 is provided in the *Proceedings of the North American Pavement Management Conference (8)*.]

There are, however, many agencies at the local government level with a diverse range of street and highway systems and with a multiplicity of requirements for effective use of the systems. The purpose of this paper is to briefly describe the results of the study reported by Monismith et al. *(7)*. This study is sponsored by the State of California Department of Transportation and the Federal Highway Administration (FHWA) to enhance the development of pavement management activities at the local government level. It is considered worthwhile to present the results of the study obtained so far; essentially it consists of a summary of what 13 local government agencies, identified generally in Table 1, are doing to implement pavement management activities in their organizations. It is hoped that this summary will prove of assistance to local government organizations who may wish to undertake pavement management activities.

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TABLE 1 ORGANIZATIONS PARTICIPATING IN STUDY

Local Government Organization	General Location
City A	Western United States
City B	Central West Coast, U.S.
County A	Southwestern United States
City C	Central West Coast, U.S.
City D	Northern West Coast, U.S.
City E	Central East Coast, U.S.
City F	Central West Coast, U.S.
City/County A	Central West Coast, U.S.
City G	Western United States
County B	Northern West Coast, U.S.
County C	Western United States
City H	Southern West Coast, U.S.
Regional Municipality A	Eastern Canada

Summarized in Table 2 is general information about the organizations whose systems were evaluated. It is included to provide a perspective on all of the information presented.

The results will be discussed within a general framework for pavement management, illustrated in Figure 1.

PAVEMENT MANAGEMENT SYSTEMS FOR CITIES AND COUNTIES

The degree of completeness of a pavement management system (PMS) can range from a simple data base to a system that includes the feature of optimization. Between these two levels there is a range in possible systems. The level required will, to a large extent, be influenced by the objectives set for the system.

Based on discussions with personnel in local government agencies visited in conjunction with this project, there appears to be a primary requirement for a PMS at the city-county level:

TABLE 2 GENERAL INFORMATION

AGENCY	POPULATION	STREET/ROAD MILEAGE Miles	BUDGET, Dollars, × 10^3			DEVELOPMENT OF SYSTEM
			Rehabilitation	Maintenance	Rehabilitation and Maintenance	
City A	90,900	286 Centerline	1,500	288	1,788	Consultant
City B	100,000	550 Centerline	500		500	APWA - PAVER System
County A	1,600,000	3,700 Centerline (1,300 Paved Centerline)				County Staff (in-house)
City C	50,000	220 Centerline	500			Consultant and City Staff
City D	500,000					Consultant and City Staff
City E	219,000	1,736 Lane	600			Consultant and City Staff
City F	85,000	200 Centerline			1,400	City Staff FHWA - CALTRANS System
City/County A	700,000	850	6,000	3,000	9,000 (1985-86)	Consultant and City Staff
City G	46,000	360 Lane				APWA - PAVER System
County B	139,000	860	1,687	678	2,365	Consultant with State and County Staff
County C	125,000	3,150 Centerline	500	5,000	5,500	County Staff FHWA-CALTRANS System
City H	80,000	260	800	75	875	City Staff
Regional, Municipality A	225,000	500 Centerline			12,000 Can.	Consultant with Municipality Staff

the system should be simple to maintain and operate. It should be noted, however, that the definition of what is simple will vary from agency to agency, depending on its size and the resources available to support a PMS. It was also indicated that user-friendly, menu-driven software is a desirable attribute of a PMS. Such a system provides interactive use for data entry, editing, and retrieval of information rapidly and easily and at remote terminals by users at various levels of management. After the requirement for simplicity, agency priorities vary somewhat. Wells, et al. *(9)*, as the result of a development program in the PMS area by the Metropolitan Transportation Commission (encompassing the nine-county area surrounding the San Francisco Bay), have listed three features as being of primary importance in a PMS for local governments, as follows:

1. A procedure to objectively quantify pavement condition,
2. A listing of the most cost-effective maintenance treatments, and
3. A means of matching treatments to problems.

General Approach

The framework of Figure 1 provides the general format for pavement management activities. The data bank is the heart of the system. Exactly what is included in the data bank will depend on the system requirements. As a minimum, information concerning the condition of pavements in the network must be obtained. Based on pavement condition, it will be necessary to establish a set of actions considered appropriate for each condition state; i.e., single or multiple variable indices.

The best treatment from the feasible set must be determined. The treatment may be obtained from a consensus of knowledgeable people, usually within the agency personnel. This "best" action can also be determined by use of prediction models and optimization procedures.

Priorities can be developed based on ranking procedures; benefit-cost ratios; maximizing performance or condition of network; minimizing cost; or other methods may be developed using performance, benefits, and cost as primary considerations. In most cases, the needs will exceed the funds available. Priorities can be used to select sections for corrective action. It should be noted, however, that corrective measures for some sections may have to be deferred to a future year.

A careful evaluation of each section will be essential to ensure that the information in the network data base is correct and that there are no site-specific conditions that would alter the plan developed for the network branch of the system.

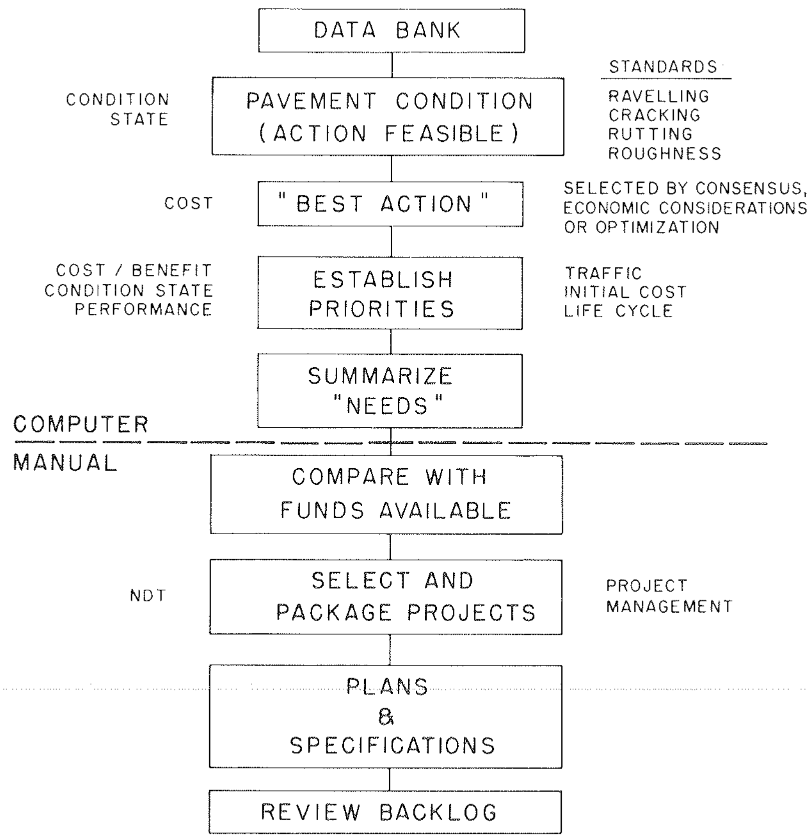

FIGURE 1 Framework for pavement management system: city and county level.

Finally, plans and specifications are prepared for implementation of the program.

Feedback is an important part of the PMS. That is, what is happening to the overall condition of the pavement network? Is it improving, deteriorating, or remaining the same? This review of the backlog of needs will be useful in requesting funds to maintain the pavement network at a desired level.

Planning a PMS

There are a number of factors to be considered in planning the development of a PMS. Some of the more important ones discussed with the agency personnel include

1. Availability of resources,
2. Information requirements,
3. Level of sophistication (completeness),
4. Data management,
5. Reporting, and
6. Administration.

Resource Requirements

Resources can be divided into three categories: *(a)* personnel, *(b)* equipment, and *(c)* funds. The resource requirements can be divided into two levels, i.e., those needed for development and those required for operation of the system.

Because of a shortage of personnel with training or background in development of PMS, most agencies have called on consultants to assist in the development process. There are exceptions, e.g., personnel in Cities *F* and *H* and County *C* have developed or adapted systems for their respective agencies. When consultants are retained, it is usually a joint effort, with the agency providing the kind of assistance for which it can be most helpful.

Cities and counties, for the most part, have not acquired equipment to be used in the field, i.e., car ride meters, deflection testers, skid testers, road loggers, and so on. Again, there are some exceptions in the case of deflection testing equipment; however, the majority of agencies rely on commercial companies to provide this type of equipment.

Computer equipment is available and is being used by all the agencies contacted. Some cities and counties have access to mainframe computers in house; some have microcomputers assigned within the department that will maintain the PMS.

Most of the agencies contacted would prefer to have their own computer, usually a microcomputer, assigned to the responsible department. In this way, the department can maintain direct control of the system, update it in a timely manner, produce reports when and of the type necessary, and interact with the data base for editing and retrieval. Generally, the system users want a user-friendly (interactive) capability and menu-driven program.

Funding is always a problem for cities and counties, both for development and operation of the PMS. As will be seen subsequently, the cost of developing a PMS can range from as little as \$10,000 to as much as \$500,000 or more (not counting data acquisition), depending on the level of sophistication required.

TABLE 3 STREET/ROAD IDENTIFICATION

AGENCY	CODE	ROUTE IDENTIFICATION	LIMITS: BLOCK-BY-BLOCK	LIMITS: MILE POST	LIMITS: OTHER	CLASSIFICATION: NUMBER	CLASSIFICATION	SPECIAL CONSIDERATIONS: LANE IDENTIFICATION	SPECIAL CONSIDERATIONS: INTERSECTIONS	SPECIAL CONSIDERATIONS: BUS LANES	SPECIAL CONSIDERATIONS: PARKING AREAS	SPECIAL CONSIDERATIONS: BICYCLE LANES				SPECIAL CONSIDERATIONS: OTHER	COMMENTS
City A	X	X	X			5	a) Principal Arterial b) Minor Arterial c) Principal Collector d) Minor Collector e) Local										
City B	X				X												
County A	X	X		X		2	a) Paved b) Unpaved										
City C	X	X	X			6	a) Collector b) Arterial c) Local Residential d) Local Business District e) Local Industrial f) Through Truck or Bus Route									X	Traffic index (State of California) associated with each classification.
City D	X	X	X			4	a) Arterials - nonFAU b) Arterials - FAU c) Collectors d) Locals NOTE: Further subdivisions used: 1) Core Area Street 2) Transit Mall Street 3) Transit (Tri-Met) Street 4) Light Rail Street 5) Light Rail/Transit Street 6) Industrial Area Street	X		X						X	
City E	X	X	X		X	8	a) Major Arterial b) Arterial c) Collector d) Heavy Industrial e) Light Industrial f) Local g) Residential h) Cul-de-sac		X		X						

TABLE 3 *continued*

AGENCY			LIMITS			CLASSIFICATION		SPECIAL CONSIDERATIONS									COMMENTS
	CODE	ROUTE IDENTIFICATION	BLOCK-BY-BLOCK	MILE POST	OTHER	NUMBER		LANE IDENTIFICATION	INTERSECTIONS	BUS LANES	PARKING AREAS	BICYCLE LANES				OTHER	
City F	X	X(1)	X		X	X	a) Arterial b) FAU Route c) Bus Route d) Collectors e) Residential	X	X			X					Sections identified by node numbers. (1) for FAU routes.
City/County A	X	X	X			4	a) Arterial - with buses b) Arterial - without buses c) Local - with buses d) Local - without buses		X	X						X	Consideration of tracks
City G	X				X	3	a) Commercial b) Primary c) Residential										Sections and 2500 $ft.^2$ sample units
County B		X		X		5	a) Principal Arterial b) Minor Arterial c) Major Collector d) Minor Collector e) Local Access	X									Segments less than or equal to 0.5 mi.
County C	X	X		X	X	3	a) Major - FAS/FAU b) Select c) Minor	X	X							X	Lengths between intersections (usually 1 mile maximum length). Roads classed as diskable.
City H		X	X			4	a) Arterial b) Industrial c) Industrial d) Residential			X						X	Design traffic level associated with each classification.
Regional, Municipality A	X	X			X		None										

In planning the PMS, a realistic estimate of the amount of funds available is very important.

Information Requirements

The three main types of data files considered by the agencies include *(a)* design and construction, *(b)* maintenance history, and *(c)* pavement condition.

The design and construction file can include information relative to parameters related to construction or reconstruction; for example, dates, traffic, soil support, materials, and layer thicknesses. More or less information can be included as desired.

The maintenance history file can include information relative to what was done to maintain a segment as well as its timing. Overlays, surface treatments, base repairs, and crack sealing are specific examples of maintenance activities. Historical information of this kind is useful to the engineer when packaging projects.

Information to be included in the pavement condition file will vary depending on local experience. Typical kinds of information for flexible pavements include surface type; transverse cracking; fatigue (alligator cracking); deformation (ruts and corrugations); edge deterioration (cracking, shoulder drop off); block cracking; patching; utility cuts; ride; and raveling.

In most cases agencies agree that they started by trying to collect more information than was necessary. This slows down the condition survey, reduces reliability of information, requires increased computer storage and programming and, in general, is nonproductive. The rule should be: Collect only what is necessary.

PAVEMENT MANAGEMENT SYSTEMS: SPECIFIC CONSIDERATIONS

When developing a PMS a number of items must be considered. These include

Section identification,
Pavement condition surveys,
Other files,
Maintenance and rehabilitation alternatives,
Performance prediction,
Network programming,
Optimization,
Data management, and
Reports.

Some of these items will be discussed in this section; they are based on results of the survey and are summarized in Tables 3 to 8.

Section Identification

Street or roadway identification is required for data collection, analysis, and reporting purposes. Codes or street names, or both, have been used for section identification. Alphanumeric codes can be used to describe street classification (arterial, collector, and so on); general location in city; maintenance responsibility; and number of issues. The codes are tied to specific street sections with defined limits. A street or road section should be a consistent pavement type [portland cement concrete (PCC), asphalt concrete (AC), and so on]; pavement structural section; and traffic volume. The beginning and ending of the sections or section limits should be clearly identifiable in the field. Table 3 provides a summary of the procedures used by the organizations interviewed.

Section Length

Some cities have elected to designate sections on a block-by-block basis. Other organizations have sections that are several miles in length. Selection of section length should give consideration to the following:

1. Uniformity of the section:

   Pavement type,
   Pavement structural section,
   Traffic volume,
   Age of pavement,
   Rehabilitation history, and
   Maintenance history.

2. Classification

   Functional (arterial, collector, residential); and
   Funding (federal, state, county, and city special funding categories).

3. Scheduling rehabilitation and maintenance activities.

As a general guide, section limits should be selected on the basis of uniformity, with consideration given to classification and scheduling. If this approach is followed, section length will not be a constant.

Classification

Most cities use a functional classification for their streets. Requirements for typical structural sections, geometrics, drainage, and so on, will be associated with the classifications used. These classifications may also be associated with funding categories. For example, major arterials may be eligible for federal funding, whereas rehabilitation of residential streets must be paid for by adjacent property owners. These considerations are important when scheduling rehabilitation and maintenance activities.

Other Considerations

The preparation of budgets requires an estimate of the area of pavement to be rehabilitated or maintained. Thus, the street or lane width, number of lanes, parking areas, and so on, in addition to section length, become important. It may or may not be necessary to measure street width. For example, if section limits are based in part on street classification and if geometrics are based on street classification, network budget preparation will not require street width to be measured as part of the inventory.

Intersections have special pavement and drainage problems that require separate considerations. These special problems can be handled by notes attached to street evaluation forms or by separate special forms. If special forms are used for intersections, the limits of the intersection need to be defined.

Considerable time and expense is often required to establish street section limits. As indicated above, construction, rehabilitation and maintenance history files, as well as traffic files, should be consulted before establishing limits. Unfortunately, many cities and counties have poor records that cannot be readily used.

Pavement Condition Surveys

Pavement smoothness (ride quality) and safety (skid resistance) are important functional considerations. Pavement distress, while important from a functional standpoint, is extremely important as an indication of structural condition.

Ride quality is usually evaluated during pavement distress condition surveys; e.g., by use of a car ride meter. Safety is often evaluated by measuring a pavement friction number or by accident frequency information, or both. These objective measuring techniques are not widely used by cities and counties at the present time. Subjective measurements of safety are sometimes made by recording the presence of flushing or bleeding.

Pavement smoothness and coefficient of friction are dependent on speed. The slower speeds of city traffic decrease the relative importance of these functional performance items compared with those of rural county or state roadway networks.

Types of Distress

Pavement structural condition surveys are performed to identify the type, extent, and severity of several distress types. Indications of permanent deformation (rutting, shoving, corrugations), surface distress (flushing, raveling, surface wear, fuel damage), cracking (alligator, longitudinal, transverse) and maintenance (patching) are usually included in these surveys (see Table 4). As seen in this table, the number of distress types and the detail to which the extent and severity of each distress type is recorded are highly variable among existing city and county pavement management systems. The detail required should be dictated by the end use of data. If selection of rehabilitation and maintenance alternatives is desired from recorded distress information, the distress condition survey forms should be developed to include the required detail. Experience suggests that the amount of detail required is usually much less than was originally envisioned.

Method of Data Collection

Condition survey information can be collected electromeechanically or by visual surveys. Visual surveys are usually performed by a single individual or a two-person team. Survey techniques require driving the street sections at a slow speed or walking along all or portions of the street section. Windshield or driving surveys may involve stopping of the vehicle for short periods of time. Walking surveys range from walking the entire length of the section to walking three to five randomly selected portions of a pavement section. Walking surveys provide more accurate data than riding surveys, but costs are higher. The degree of accuracy required of the data should be considered when selecting the data-collection methodology.

A variety of electromechanical devices are available to record pavement distress. These devices range from units with electronic input for recording data as they would appear on a manual input form to the use of enhanced video images to record the type, degree, and extent of selected types of pavement distress. None of the existing PMS reviewed for this study presently uses electromechanical devices.

Frequency of Surveys

Condition surveys should be performed at sufficient intervals to monitor pavement condition changes that will affect selection of rehabilitation or maintenance alternatives. Roadways in relatively poor condition may need to be surveyed annually, whereas new sections with relatively good performance can be evaluated every 2 to 3 years. Usually 1/3 to 1/2 of all pavement sections are evaluated annually. Thus, a 2- to 3-yr cycle is considered adequate by many organizations.

Condition surveys are most often performed in the spring. Current condition information is then available to assist in scheduling summer rehabilitation and maintenance activities. In addition, roadways are usually in their worst condition during the spring. However, some cities will perform condition surveys in the winter and summer.

Condition Survey Index

Condition survey information can be used together with an appropriate scoring system to develop a condition index. Most systems assign deduct points to specific types, degrees, and extents of distress. These deduct points are summed and subtracted from 100. This process results in a single value index to describe condition index.

Several systems used this pavement distress in combination with other numeric scores, such as roughness, drainage, and so on, to calculate a combined score, which is used to describe the roadway or street condition.

Supporting Information

In addition to the data and information associated with pavement condition, most pavement management systems provide for supporting data to be used for such things as priority scores, engineering analysis, and cost data. Table 5 provides a summary of the other general types of data collected.

Files for the supporting data are often referred to as *(a)* design and construction, *(b)* maintenance history, *(c)* rehabilitation and maintenance, *(d)* drainage, and *(e)* geometrics. There may be others; the above are mentioned as illustrations.

To illustrate further, the design and construction file would contain information to relate design to life-cycle performance; maintenance history would provide information on the sequence of actions (treatments) subsequent to construction and life-cycle evaluation. These supporting files could also provide

TABLE 4 CONDITION SURVEYS

AGENCY	PHYSICAL DISTRESS: TYPE																							PHYSICAL DISTRESS: METHOD OF DATA COLLECTION				PHYSICAL DISTRESS: FREQUENCY		COND. INDEX		ROUGHNESS		Deflection	SAFETY			COMMENTS
	RUTTING	SHOVING	CORRUGATION	FLUSHING	RAVELING	SURFACE WEAR	FUEL/DRIPPING DAMAGE	ALLIGATOR CRACKING	LONGITUDINAL CRACKING	EDGE CRACKING	TRANVERSE CRACKING	BLOCK CRACKING	SLIPPAGE	POTHOLES	PATCHING - THIN	PATCHING - THICK	UTILITY TRENCH	POLISHED AGGREGATES	REFLECTED CRACKING	SEALS	SPALLING	SLAB BREAKUP	FAULTING	VISUAL - WALKING	VISUAL - WINDSHIELD	ELECTRONIC	MECHANICAL	PERCENT SECTIONS PER YR.	TIME OF YEAR**	SINGLE VALUE	COMBINED VALUE	SUBJECTIVE	OBJECTIVE	OBJECTIVE	SUBJECTIVE	OBJECTIVE - FRICTION NO.	OBJEC. - ACCIDENT FREQ.	* - See final page ** of table.
City A	X	X				X		X	X		X			X		X	X								X			50	W Sp			X						
City B	X	X	X	X	X			X	X	X	X	X	X	X	X	X	X	X	X					X				10			X	X			X			
County A	X	X	X	X	X	X		X	X		X			X				X						X				50			X	X			X		X	
City C	X		X		X			X				X		X**											X			50 50	M A O N	X								** - Special Report
City D	X				X			X	X		X		X												X			*	Jan Jul		X	X		**				* - 50% Arterials 30% Locals ** - Dynaflect Deflections
City E			X	X																					X			33	W Sp	X		X			X			Relative performance age evaluated.
City F	X				X	X	X	X	X		X	X		X	X	X	X			X				X	X			*		X		X		**			X	* - Sections not receiving Maint./Rehab. surveyed yearly. ** - Deflections meas. with Dynaflect on entire system
City/County A					X			X	X		X	X													X			50	Sp	X	X	X*		**				* - Contribution to ride qual. made by RR tracks is assessed. ** - Only when structural evaluation required.

TABLE 4 *continued*

AGENCY	PHYSICAL DISTRESS — TYPE																							METHOD OF DATA COLLECTION				FRE-QUEN-CY		COND. INEX		ROUGH-NESS		Deflection	SAFETY			COMMENTS
	RUTTING	SHOVING	CORRUGATION	FLUSHING	RAVELING	SURFACE WEAR	FUEL/DRIPPING DAMAGE	ALLIGATOR CRACKING	LONGITUDINAL CRACKING	EDGE CRACKING	TRANSVERSE CRACKING	BLOCK CRACKING	SLIPPAGE	POTHOLES	PATCHING - THIN	PATCHING - THICK	UTILITY TRENCH	POLISHED AGGREGATES	REFLECTED CRACKING	SEALS	SPALLING	SLAB BREAKUP	FAULTING	VISUAL - WALKING	VISUAL - WINDSHIELD	ELECTRONIC	MECHANICAL	PERCENT SECTIONS PER YR.	TIME OF YEAR**	SINGLE VALUE	COMBINED VALUE	SUBJECTIVE	OBJECTIVE	OBJECTIVE	SUBJECTIVE	OBJECTIVE - FRICTION NO.	OBJECTIVE - ACCIDENT FREQ	
City G	X	X	X	X	X			X	X	X	X	X	X	X	X	X	X	X						X				10		X		X			X			
County B			X	X	X			X	X	X	X				X	X					X	X	X	X				50			X	X						
County C	X			X	X			X	X		X	X		X	X	X	X			X				X				33	Jan Jul	X	X	X		*				* - AC overlays assigned based on deflection data.
City H					X			X	X			X			X	X	X		X	X	X	X		X	X			*			X	X		**				* - Worst 1/3 of system 1st year; 4 yr. sequence then followed ** - Deflections meas. on cand. proj. using FWD
Regional, Municipality A	X	X	X	X	X	X		X	X	X	X		X											X				50	Sp Su	X	X		X					

** - Key: W = winter, Sp = spring, Su = summer, F = fall; M = March, A = April, Jan = January, and Jul = July.

TABLE 5 OTHER DATA

AGENCY	HISTORIC					DRAINAGE				TRAFFIC				GEOMETRICS						OTHER				
	INITIAL DESIGN	SURFACE TYPE	CONSTRUCTION	REHABILITATION	MAINTENANCE	OVERALL	CURB AND GUTTER	SIDE DITCHES	VALLEY GUTTER	ADT	PERCENT OF TRUCKS	EAL/TRAFFIC INDEX	PEAK HR. TRFC. EST.	CROSS SLOPE	LONGITUDINAL SLOPE	NUMBER OF LANES	SEGMENT LENGTH	WIDTH	AREA	SIDEWALKS	UTILITIES	SPEED LIMIT	DIRECTION	LAND USE
City A	X		X	X	X		X		X	X								X	X	X	X			
City B																		X	X					
County A		X		X		X				X						X	X	X	X			X	X	
City C	X	X	X	X	X		X			X		X				X	X	X	X					X
City D		X	X	X	X	X	X			X	X					X	X	X					X	
City E						X										X		X	X					
City F										X	X					X	X	X	X			X	X	
City/County A	X	X	X	X	X		X			X		X			X	X	X	X	X					
City G		X	X		X		X					X				X	X	X	X					
County B	X	X	X	X	X					X	X		X			X	X	X	X					
County C		X	X	X	X	X				X						X	X	X				X	X	
City H		X	X*	X*	X*					X	*	X				X	X						X	X**
Regional, Municipality A	X				X					X		X				X		X	X					

* - Data gathered for segments during project design, not for entire network as yet.

** - Basis for classification of non-arterial or non-collector streets.

information required to calculate benefit-cost ratios based on such considerations as number of users, e.g., average daily traffic (ADT), type of users (equivalent axle loads), and the maintenance-plus-rehabilitation costs.

Drainage information, which could influence performance, may be included as a special file or in the design and construction file. Geometrics, signs, legends, and so on, could be added, provided some benefit or use could be identified.

One of the major problems in developing a PMS is the tendency to collect more information than is necessary or useful for the system. Experience indicates that care should be taken not to collect more information than is necessary to support the system. Thus, in developing a PMS, every bit of information to be collected should pass the following test as a minimum:

1. The data will be used to identify sections with poor performance,
2. The data will be used to establish priorities,
3. The data will be used to select maintenance or rehabilitation actions,
4. The information will be used to calculate the cost of maintenance and rehabilitation actions,
5. The information can be used to estimate life-cycle costs of each maintenance and rehabilitation action, and
6. The information can be used to estimate the life-cycle costs of newly constructed pavements.

Additional information can be selected; however, the criteria for selection should include how the information is to be used in the PMS or some other system useful to the agency. Avoid collecting information that would be "nice to have."

Maintenance and Rehabilitation Alternatives

Table 6 shows the types of rehabilitation and maintenance alternatives used by agencies surveyed in this study. From 3 to 16 alternatives are used in the various systems.

Performance Prediction or Network Programming (Data Utilization)

Figure 2 provides an illustration of the various levels of a PMS that can be produced, starting with a data base of information

TABLE 6 REHABILITATION AND MAINTENANCE ALTERNATIVES

AGENCY	THICK OVERLAY	INTERMEDIATE OVERLAY	THIN OVERLAY	THICK OVERLAY & INTERLAYER	INTERMEDIATE OVERLAY & INTERLAYER	THIN OVERLAY & INTERLAYER	MILL & FILL	MILL, FILL & OVERLAY	HEATER-SCARIFICATION & OVERLAY	IN-PLACE RECYCLING	CHIP SEAL	SLURRY SEAL	CAPE SEAL	SPECIAL BINDER SEAL	FOG SEAL	REJUVENATOR SEAL	RECONSTRUCTION	THIN OVERLAY & LOCAL DIGOUTS	THIN OVERLAY & LEVELING	INTERMEDIATE OVERLAY & LOCAL DIGOUTS		CRACK SEALING	SURFACE PATCHING	REMOVE & REPLACE PATCHING		DISK AND RELAY	HEAT, DISK, AND RELAY	COMMENTS
City A		X				X	X		X		X						X						X	X				
City B																												
County A	X	X	X						X	X	X	X			X	X	X					X	X	X				NO presently considered in system
City C						1-1/2" X	X					X	X				X						X	X				
City D			X								X	X					X					X		X				
City E	X	X	X						X		X	X	X		X							X	X	X				
City F		X	X			X	X			X	X	X	X		X		X	X	X			X	X	X				
City/County A		X			(1)		X	X				X					(1)	(1)	(1)									(1) Alternatives considered by design engineer.
City G	X							X	X	X	X				X							X		X				Other alternatives considered by PAVER
County B	X		X								X	X																Alternate rehab strategies may be considered.
County C	X	X									X					X						X	X	X		X	X	Also uses sand seal
City H	X	X	X	X	X	X			X	X		X			X	X	X			X		X	X	X				
Regional, Municipality A		X	X														X											

TABLE 7 DATA UTILIZATION

AGENCY	Basis for Prediction Models					Network Programming						Network Optimization Objective Function		Data Management Hardware				COMMENTS
	EXPERIENCE DATA	HISTORIC DATA - REGRESSION	HIST. DATA - TRANSITION MATRIX	SUPPLIED BY SYSTEM	MECHANISTIC	RANKING BY SCORES	PRIORITIES	AGENCY	CONSULTANT	AVAIL. TO OTHER USERS	PROGRAM DEVELOPED BY	MINIMIZE COSTS	MAXIMIZE BENEFITS	CARD	PERSONAL COMPUTER	MAINFRAME	PACKAGING CAPABILITY	
City A						X	X			X	Consultant	X				X		No prediction models
City B		X		X		X			X		APWA-COE					X		Prediction models not used
County A		X									In-house				X			Changed from mainframe to PC/AT in 1985
City C	X						X		X	X	Consultant				X	X		Initially on mainframe (1979) Changing to PC (1985)
City D	X					X			X		Consultant					X	X	
City E				X			X		X		Consultant					X		Priorities based on type of treatment - heavy overlays first
City F							X			X	CALTRANS-FHWA					X		Prior. based on cost effec. treat. Pvmts with high deflect. get first priority.
City/County A	X						X				Consultant				X	X		Changing to PC during second year
City G				X		X			X		APWA-COE					X		
County B		X				X		X		X	Washington DOT Consultant	X			X	X		Input and output optional on PC - manipulation and data storage on mainframe
County C	X					X	X			X	CALTRANS FHWA				X			
City H						X	X			X	In-house					X		
Regional, Municipality A			X			X	X		X		Consultant	X				X		

TABLE 8 REPORTS

AGENCY	STREET/ROAD LISTING	RANKING: CONDITION SCORE	RANKING: PRIORITY SCORE	RANKING: DISTRESS TYPE	RANKING: REHAB/MAIN ALTERNATIVES	RANKING: MAINT. AREA/DISTRICT	RANKING: RIDE QUALITY	RANKING: ROAD/STREET CLASSIF.	BUDGET: BY YEAR	BUDGET: BY REHAB/MAINT. ALTERN.	BUDGET: 3 - 5 YEARS	OTHER REPORTS: CURB & GUTTER CONDITION	OTHER REPORTS: SLIDE DUTCH CONDITION	OTHER REPORTS: VALLEY GUTTER CONDITION	OTHER REPORTS: SIDEWALK CONDITION	OTHER REPORTS: SAFETY	OTHER REPORTS: DESIGN & CONSTRUCTION	OTHER REPORTS: MAINTENANCE HISTORY	OTHER REPORTS: COST ANALYSIS
City A	X	X		X	X	X	X		X			X		X	X				
City B	X	X			X				X		X								
County A	X	X			X	X	X	X										X	
City C	X																	X	
City D	X	X						X	X										
City E	X	X			X				X	X	X								
City F	X	X(1)	X	X	X			X	X										X
City/County A	X	X	X	X					X	X							X	X	
City G	X	X			X				X		X								
County B	X	X	X	X	X	X		X	X	X	X							X	
County C	X		X		X				X								X	X	
City H	X	X	X	X					X										
Regional, Municipality A	X	X	X						X		X								

(1) By Dynaflect analysis, rater score.

(6). The five components of the data base are construction history, inventory, traffic, surface friction, and pavement condition.

It is not necessary to include all of the files indicated above; however, some agencies may want to add additional files for maintenance history, signing, drainage, shoulders, and so on.

The key information for most city and county systems will be contained in the traffic, construction history, and condition-rating files. Depending on what information is contained in the construction history file, a record of maintenance may be useful and may be incorporated in a separate file. With these data, various methods of use are available. These are summarized in Table 7 for the organizations involved.

The condition file can be used to evaluate the overall health of the pavement network by a simple tabulation of condition, as illustrated in Figure 3, taken from reports prepared for the city of Vacaville, California (not included in this study). In this report, each street segment has been ranked according to the severity and extent of fatigue (alligator) cracking. Severity ranges from 1 to 3 (with 3 being the most severe) and extent ranges from 1 to 4, according to the percent of the length affected (4 being the greatest extent). Thus, if fatigue cracking is considered to be the most critical condition rated, the first nine sections listed would be given first consideration for corrective action. Other statistical information could easily be produced from this type of information. For example, what percentage of the segments in the network have fatigue cracks in severity level 3 and extent 4?

The interpreting program referred to in Figure 2 translates (interprets) the information into a combined rating for each section using condition data in the data base. This is accomplished by applying weighting values to the extent and severity of each distress category.

The combined index can be used in a number of ways:

1. To establish priorities;
2. To summarize overall condition of pavements in the network, i.e., health of system;
3. To develop performance curves (predictions) over time; and
4. To provide performance trends based on budget level.

Priorities are necessary to determine which projects to rehabilitate when there are funding constraints. However, before priorities can be established it is necessary to identify which segments need rehabilitation or maintenance.

Figure 4 illustrates a technique for establishing threshold values for maintenance or rehabilitation. Using this method,

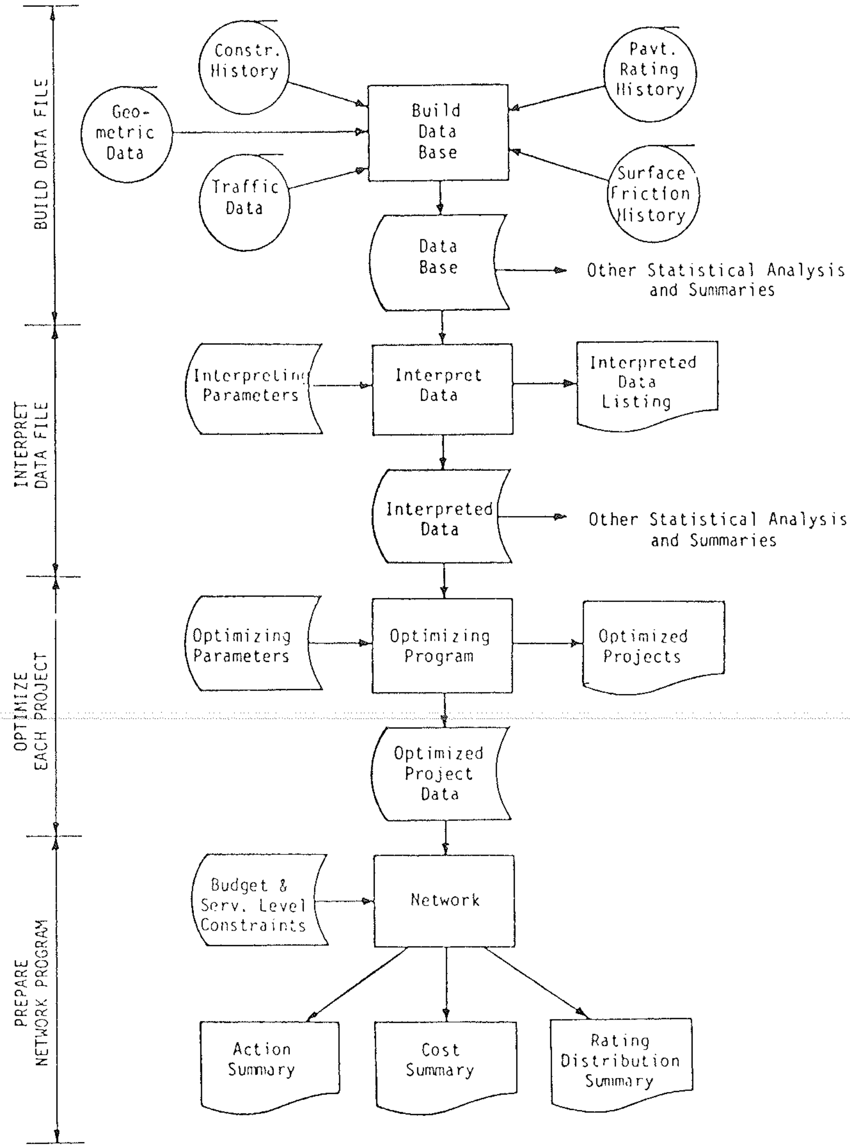

FIGURE 2 Conceptual flow chart of Washington State operations *(6)*.

deduct values are assigned to each type of distress to be remedied by treatment. Depending on the condition and traffic (as measured by the Traffic Index, TI), an action is recommended. If more than one deficiency is noted, a selection logic is used to define the action that will correct all recorded deficiencies below the threshold value, as shown in Figure 5. This latter technique is similar to procedures used by cities and counties that use the California Department of Transportation PMS *(6)*.

Thus, priorities can be established without prediction models or optimization. A procedure of this type is relatively simple; however, it relies almost entirely on engineering judgment and experience to identify threshold values and the best action or treatment. In many cases, this is all that is needed. If prediction models are available, it is possible to develop an optimization procedure. Only a limited number of systems for cities and counties incorporate such procedures. The general approach is to develop some objective function that is to be minimized or maximized; for example, costs can be minimized, or the ratio of benefits to costs can be maximized. Virtually all of the techniques used depend on some measure of performance and cost.

The benefits of optimization are considerable, providing the resources can be made available to develop and maintain such a system. Optimization will compare a large number of alternative treatments for a pavement section in such a way as to recommend the best action and the best time to initiate a treatment, depending on prevailing conditions.

Regardless of the level of sophistication incorporated into the system, the final step is to package the projects into a program for development of a set of plans and specifications, and eventually an action plan.

Reports

Existing data-management programs provide an opportunity for the user to sort and provide data in numerous report formats. Street listings, rankings, and budget information are examples of the reports provided by all surveyed PMS. Table 8 contains a summary of the various kinds of reports generated.

SECT	STREET	FROM	TO	DATE OF SURVEY	LNGTH	WIDTH	LNS	ALGT CRCK SEV	ALGT CRCK EXT	EDGE CRCK SEV	EDGE CRCK EXT	RAVL SURF SEV	RAVL SURF EXT	PAVM DIST SEV	PAVM DIST EXT	POTH
130	ALAMO DRIVE	VANDEN ROAD	LEISURE TOWN ROAD	05/28/85	0.65	25	2	3	4	3	2	2	1	1	2	1
420	ELMIRA ROAD	NUT TREE ROAD	CHRISTINE DRIVE	05/01/85	0.25	24	2	3	4	3	4	1	1	1	1	0
430	ELMIRA ROAD	CHRISTINE DRIVE	EDWIN DRIVE (WEST)	05/01/85	0.20	45	2	3	4	2	4	0	0	0	0	0
440	ELMIRA ROAD	EDWIN DRIVE (WEST)	LEISURE TOWN ROAD	05/01/85	0.45	60	2	3	4	0	0	0	0	0	0	0
470	LEISURE TOWN ROAD	CITY LIMIT	INTERSTATE 80	05/29/85	0.10	25	2	3	4	2	2	2	1	1	2	1
480	LEISURE TOWN ROAD	INTERSTATE 80	ULATIS CREEK	05/29/85	1.25	25	2	3	4	3	2	3	1	1	2	1
500	LEISURE TOWN ROAD	KINGSWOOD AVENUE	ALAMO DRIVE	05/29/85	0.70	25	2	3	4	3	3	2	1	1	2	1
650	MERCHANT STREET	ELM STREET	ALAMO DRIVE	05/01/85	0.30	45	2	3	4	0	0	1	1	1	1	0
950	SEQUOIA	LEISURE TOWN ROAD	YELLOWSTONE DRIVE	05/29/85	0.10	50	2	3	4	3	2	3	1	0	0	1
70	ALAMO DRIVE	CRYSTAL LANE	ALDERWOOD WAY	05/28/85	0.30	30	2	3	3	0	0	0	0	1	1	0
670	MERCHANT STREET EB	STEVENSON STREET	MAIN STREET	05/01/85	0.30	25	4	3	3	0	0	1	2	0	0	0
675	MERCHANT STREET WB	MAIN STREET	STEVENSON STREET	05/01/85	0.30	25	4	3	3	0	0	1	2	0	0	0
750	MONTE VISTA AVENUE	JUST WEST OF INTERSTATE 505	ROAD CLOSED	05/30/85	0.60	30	2	3	3	0	0	3	2	1	2	1
1020	YELLOWSTONE DRIVE	SEQUOIA DRIVE	NUT TREE ROAD	05/01/85	1.00	40	2	3	3	9	2	0	0	0	0	0

FIGURE 3 Listing of streets by severity and extent of alligator cracking, Vacaville, California *(7)*.

RIDE QUALITY

RATING	Deduct Values	TREATMENT TI 5	6	7	8
Acceptable	0	DN	DN	DN	DN
Tolerable	25	DN	DN	DN	OL
Unacceptable	60	OL	OL	OL	OL

AREAL CRACKING

SEVERITY	Deduct Values Extent (% of Area) N/O	1-25	26-50	>50	TREATMENT Deduct Values	TI 5	6	7	8
Acceptable	0	5	10	15	<19	DN	DN	DN	DN
Tolerable	0	13	19	26	19--23	DN	P	P	M&F
Unacceptable	0	23	30	40	26--30	OL	M&F	M&F	MF&O
					40	M&F	M&F	MF&O	RES

RAVELING

SEVERITY	Deduct Values EXTENT (% of Area) N/O	1-25	26-50	>50	TREATMENT Deduct Value	TI 5	6	7	8
Acceptable	0	4	8	12	<16	DN	DN	DN	DN
Tolerable	0	11	16	21	16--18	DN	SS	SS	OL
Unacceptable	0	18	24	31	21--31	SS	SS	OL	OL

ABBREVIATIONS

The following abbreviations are used to describe the severity and extent of the several distresses, and the recommended treatment.

CpS - Cape Seal
DN - Do Nothing
M&F - Mill & Fill*
N/O - Not Observed
P - Patch
SS - Slurry Seal
* - Includes recycled materials

CS - Crack Sealing
FS - Fog Seal
MF&O- Mill, Fill* & Overlay
OL - Overlay
RES - Restore or Reconstruct
TI - Traffic Index

FIGURE 4 Deduct values for pavement distress and recommended treatments *(7)*.

Street Listing

Street listings can be presented by sections for the entire street or for an area or district of the city by functional classification. These listings are useful as ready reference for present pavement condition and for location of sections for the annual pavement distress surveys.

Ranking

Ranking reports can be prepared based on a pavement distress condition score and from priority scores that may combine distress condition score, traffic, drainage, and so on, into a single numeric value. Reports that list roadway segments by distress type, ride quality, and rehabilitation or maintenance alternatives provide an opportunity for ranking actions. For example, all roadways with severe alligator cracking may be the highest priority for rehabilitation. Rankings can be provided within maintenance area or district or within functional classifications.

Budget

Reports describing yearly budget requirements are provided by all surveyed PMS. A few systems forecast budget requirements over a 3- to 5-yr period. Budget requirements by rehabilitation and maintenance alternative are provided by some systems on a routine basis.

COST AND PERSONNEL CONSIDERATIONS

Development and annual costs of the 13 systems, together with their personnel and equipment requirements, are summarized in Table 9. Costs to develop ranged from less than $50/mi to more than $400/mi. Values on the order of $100 to $300 appear to represent a reasonable range that might be anticipated by an organization. Generally, if little historical data are available, the costs will be on the higher side, e.g., City *D*.

Annual costs of about $100/mi to operate the system are considered to be average (see Table 9). This amount should provide a reasonable guide for budget-estimating purposes.

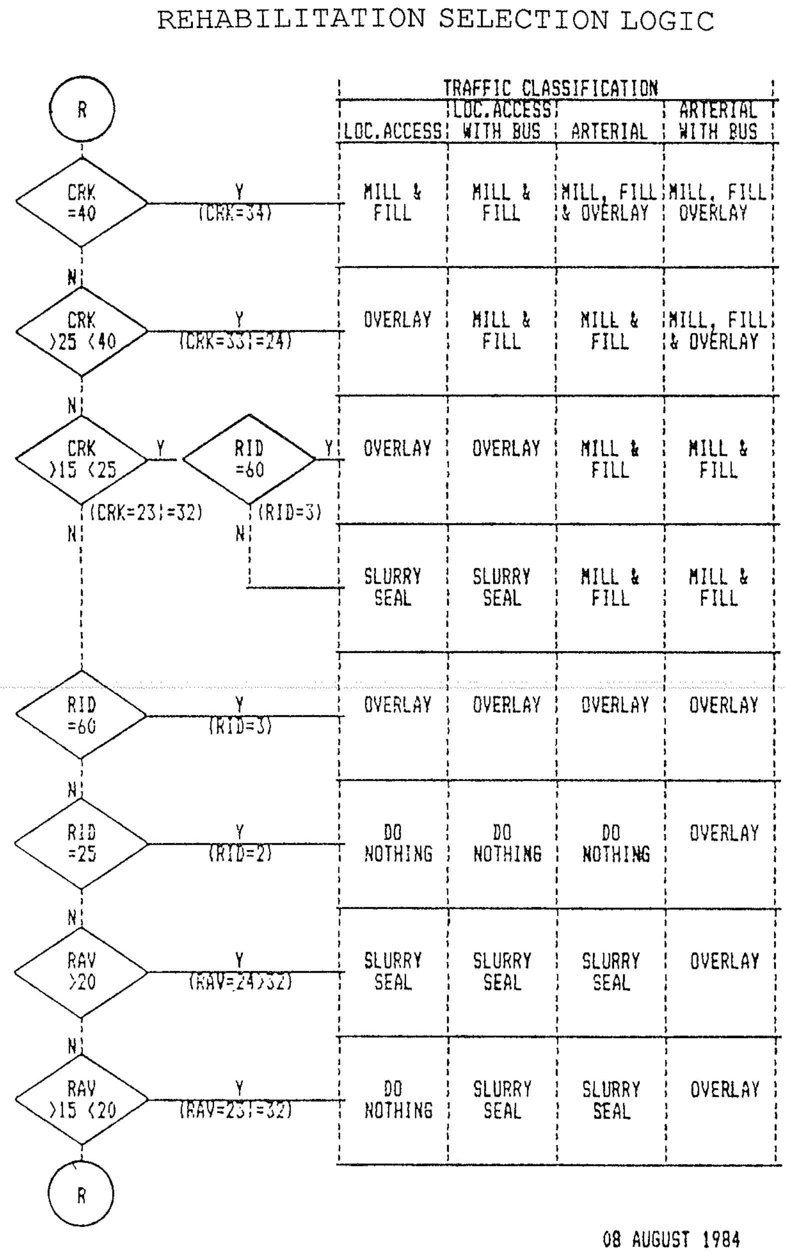

FIGURE 5 Rehabilitation logic *(7)*.

Personnel requirements will vary depending on the size of the system and whether the system is developed in house or by consultants. For smaller cities it would appear that a minimum of 2–3 person months are required to conduct the necessary activities associated with the system once it is operational. This requirement will increase as the size of the system increases, see Table 9. Regardless of the size of the system, however, one engineer within the organization should be responsible for and fully knowledgeable about the system.

As previously noted, and seen in Table 9, other than computer hardware, little additional equipment is required by the majority of the agencies surveyed. For decisions at the project level, after projects have been packaged (Figure 1) and before final designs are set, a number of organizations use the services of other organizations (e.g., consultants) to conduct structural evaluations to ensure that the treatments selected are correct.

CONCLUDING REMARKS

As funding becomes increasingly constrained, systematic procedures to assist in the decision process for fund allocation for pavement maintenance and rehabilitation activities assume greater significance. Pavement management systems, if properly formulated, can be of immeasurable assistance in this decision process to engineers responsible for road and street networks. To be effective, however, a commitment must be made to maintain and update the system and to follow the strategies proposed.

In the organizations surveyed in this investigation, the representatives stated that their systems were most helpful to them in making maintenance and rehabilitation decisions and that they planned continued use of the systems.

TABLE 9 COST AND EQUIPMENT REQUIREMENTS

AGENCY	DEVELOPMENT														ANNUAL								COST PER MILE		COST PER MILE		COMMENTS
	COST, $ × 10^3			MAN-MONTHS				EQUIP. REQUIREMENTS								COST. $ × 10^3			MAN-MONTHS								
	TOTAL	MANPOWER	EQUIPMENT	SURVEY DATA	DATA MANAGEMENT	SUPERVISION	TOTAL	VISUAL CONDITION	ROUGHNESS	DEFLECTION	FRICTION	CURING/BORING	COMP. HARDWARE	SOFTWARE	% EVAL. ANNUALLY	TOTAL	MANPOWER	EQUIPMENT	SURVEY DATA	DATA MANAGEMENT	SUPERVISION	TOTAL	DEVELOPMENT	ANNUAL	DEVELOPMENT	ANNUAL	
City A	$ 30 CONS							X							50	$ 18	18		4	1	1	6	115	60			
City B	167 CONS							X							10	20 CS											Software costs approx. $5,000 per year.
County A	50-60							X							50	40-50			9	3	1	13	45	25			Future annual costs expected to be about $30,000 - $35,000 for 1,300 centerline miles of roads
City C	50	$48	$ 2	2	4	1	7	X					X	X	100	18	$10-CS 2-DE, 6-Supervision		1.5	1	2	4.5	227	90	1.00	.40	
City D	300*		46					X		X		X			See Tbl 4	***	**		18			****					*Development costs: $ 98,000 Consultants 200,000 In-house **Data entry and data processing costs: $12,000 - $17,000 ***Personnel costs are approx. 80 percent of total ****2 engineers and 1 technician devote part-time to system
City E	39 CONS							X				X			33	30			12	3	3	18	35*	35			*Consultant's charges

TABLE 9 *continued*

AGENCY	DEVELOPMENT: COST, $ × 10^3: TOTAL	MANPOWER	EQUIPMENT	MAN-MONTHS: SURVEY DATA	DATA MANAGEMENT	SUPERVISION	TOTAL	EQUIP. REQUIREMENTS: VISUAL CONDITION	ROUGHNESS	DEFLECTION	FRICTION	CURING/BORING	COMP. HARDWARE	SOFTWARE	% EVAL. ANNUALLY	ANNUAL: COST, $ × 10^3: TOTAL	MANPOWER	EQUIPMENT	MAN-MONTHS: SURVEY DATA	DATA MANAGEMENT	SUPERVISION	TOTAL	COST PER MILE: DEVELOPMENT	ANNUAL	COST PER MILE: DEVELOPMENT	ANNUAL	COMMENTS
City F	$ 11	6	5	3	6	6	15	X	N/A	(1)	N/A	(1)	X	X	See Tbl 4	$ 7	4	3	1	1	1	3	50	35	100	78	(1) By consultant
City/County A	400*		#30**	6	3	9	18	X					X	X	50	150	$60-CS 20-DE 30-DP						470	176	.57	.21	*Development Costs: $ 56,000 Consultants 300,000 In-house **Duplicate computer hardware in maintenance and engineering sections.
City G	15-20							X							10	30-35						12	100	200			One full-time employee performs all functions
County B	5	5			1.5	1	2.5	X					X	X	50	20	15	5	2	2	1	5	6	22	.03	.15	See (a) below
County C	40	30	10	2	8	2	12	X					X		*	16	15	1	4	1	1	6	13	5	.32	.13	*Once every 3 years.
City H	12.5							X							See Tbl 4		2.6 per 1000 segments		0.6	0.6							
Regional, Municipality A	250 CONS*							X	X			X			50								530 345	30 **			*Development cost for city: $95,000 **Consultant performs update every 3 years

(a) Low development cost due to availability of a compatible road log data test.

Finally, it must be noted that the user of the PMS should remember that the system cannot make decisions, only the responsible authority can do that. Thus, professionals will always be required to properly formulate and effectively use a pavement management system.

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A Comprehensive Ranking System for Local Agency Pavement Management

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Pavement management systems for local agencies (cities and counties) require a method to determine how to allocate funds for maintenance and rehabilitation of pavements. This should provide a reasonable analysis of the impact of budget decisions on the pavement network condition and future budget needs. However, most local agencies have limited funds to collect data concerning their pavements as well as maintain them. An approach has been developed that uses a minimum of information to make reasonable budget analysis concerning maintenance and rehabilitation needs with unconstrained funding. Described in this paper is the way in which funding needs are then allocated when funding is less than needs. It includes consideration of the condition of the pavement, change of condition over time, cost of the maintenance or rehabilitation over time, and stopgap maintenance generated by deferring maintenance. This was accomplished by making it simple for the public works personnel to visualize and use. It is part of a network-level microcomputer-based pavement management system developed for San Francisco Bay Area agencies.

Much has been published recently describing how cities and counties responsible for maintaining local roads and streets *(1–3)* have far more pavement funding needs than they can meet. There are 3.9 million miles of roads and streets in the United States. Local government agencies have jurisdiction over 2.8 million miles *(4)*. Many of these pavements are nearing the end of their design life at the same time that the agencies are receiving less real financial support than in previous years.

To achieve this miracle, pavement management systems are presented as providing assistance for pavements. In general, the effort is directed at better identifying the needs, examining alternatives, and allocating available funds to provide the taxpayer with the best pavements for the funds invested. The greatest need in local agencies is for assistance in planning and programming maintenance and rehabilitation.

The basic elements of a pavement management system (PMS) directed at maintenance and rehabilitation of pavements include *(5)*:

1. A network inventory,
2. A data base,
3. Analysis procedures for network-level management, and
4. Analysis procedures for project-level management.

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The network-level management procedures are directed at planning and programming. The basic goals are to determine how much funding is needed for a given analysis period, which sections of the pavement network need maintenance or rehabilitation, and the impact on pavement condition of different funding levels. A procedure can also be provided to identify which sections need reinspection in each year of the analysis period.

The project-level procedures include the detailed engineering analysis to determine the best maintenance or rehabilitation treatment to be applied to a specific section of pavement identified as needing maintenance or rehabilitation in the network-level analysis. It is the engineering required before the development of plans and specifications, including identification of feasible alternatives and selection of the most desirable maintenance or rehabilitation alternative given present constraints.

BACKGROUND

This study was sponsored by the Metropolitan Transportation Commission (MTC), Oakland, California, and conducted through a contract with ERES Consultants, Inc., of Champaign, Illinois. MTC is the transportation planning agency for the 103 cities and counties in the San Francisco Bay Area. The origins of the PMS efforts by MTC are found in a 1982 MTC study to develop support for an increase in gasoline tax to fund local improvements for pavements *(1)*. They completed a study of local road and street maintenance needs and revenue shortfalls in the San Francisco Bay Area that indicated that local jurisdictions in the Bay Area were spending only 60 percent of funds in 1982 required to maintain roads in a condition considered adequate.

The results of this study prompted several Bay Area public works directors to ask MTC to assist them with an analysis of how Bay Area agencies could improve pavement maintenance and rehabilitation techniques and practices. This group strongly emphasized that simplicity was the most important characteristic to be included in a PMS if it were to be adopted and used by Bay Area cities and counties. They further recommended incorporation of only tried and proven techniques and practices that their staff personnel could understand and use. Finally, they indicated that the system must match the needs and resource capabilities of the jurisdictions.

In 1983, ERES Consultants, Inc., was retained to assist MTC in determining Bay Area PMS needs, PMS resources, and problems. In addition, they were to develop three basic elements of a standardized prototype PMS: a pavement condition

index (PCI), effective maintenance treatments for the Bay Area, and a network-level assignment procedure. An extensive survey was completed to determine the status of PMS implementation as well as maintenance and rehabilitation needs in San Francisco Bay Area cities and counties. Most agencies perceived a need for better management support tools; however, most of these agencies generally believed that available systems were either too complex or did not provide adequate assistance. The data currently available to the cities and counties are very limited and it is believed that they cannot afford to collect such additional data. The results of these efforts are documented in a three-volume report available from MTC *(6)*.

On the advice of the participating agencies, MTC then decided to support the development of a PMS to meet the needs of these agencies. The committee suggested that the developed system use components from existing systems as much as possible while customizing the system to specific needs. They considered the resources available for developing and implementing pavement management systems by Bay Area cities and counties along with the commitment required by the available pavement management systems. The following objectives were set:

1. Network-level capabilities for scheduling maintenance and estimating budget needs,
2. Network-level prioritization for scheduling cost-effective maintenance and rehabilitation,
3. Network-level budget estimates for alternative performance levels, and
4. Capability to be expanded to meet project-level management requirements at a later time.

A unique three-way partnership was formed to develop the pilot PMS. ERES Consultants, Inc., was retained to provide continued technical assistance for the project, with R. E. Smith as the principal investigator on the project. MTC provided most of the funding, programming expertise, and staff time to assist the participating agencies. Six Bay Area agencies, including three counties and three cities, participated in the pilot program. They provided key experienced personnel for user guidance in the development of each pavement management module and tested each as it was developed. This provided continual feedback to the developers and programmers of the system.

Determination of Budget Needs and Identification of Sections Needing Maintenance and Rehabilitation

One of the primary purposes of a pavement management system is to identify budget needs for current and future years to maintain the pavement network in an acceptable condition. In the Bay Area PMS, the analysis period was selected to be 5 years based on the normal budget procedures used by the pilot agencies. The pavement network is divided into relatively uniform segments that are expected to be given the same maintenance or rehabilitation treatment. These are then used as the basic management units in the analysis.

A modified form of the PAVER surface observable distress-based pavement condition index (PCI) was selected as the measure condition of the pavement management units and network *(5, 7)*. The distress types used to determine the PCI were reduced to the seven that were most prevalent as well as used in decision making by the Bay Area public works personnel. The distress collection was simplified by decreasing the detail required during collection. The condition in terms of PCI is projected into the future using a family curve concept adjusted for the performance of individual management units *(6)*. The condition of management units must then be connected to a maintenance and rehabilitation cost at a designated period. The funds needed for each management unit are calculated and summed for each year of the analysis period to determine network budget needs for each year.

Several approaches were considered for developing budget needs. A two-step approach was adopted. First, the most cost-effective level at which to maintain the pavements is determined in terms of cost/year of acceptable pavement life. Then the most cost-effective maintenance and rehabilitation strategies are determined to apply to the pavements at designated lower pavement condition levels. The general goal of this approach is to apply maintenance and rehabilitation at the most cost-effective condition level and return all pavements in conditions worse than this to the appropriate level based on unconstrained funding. Then, when funds are limited, an analysis is employed to select those that will be funded to provide the best network condition.

This approach required

1. Identification of maintenance and rehabilitation treatments that the Bay Area public works personnel would consider applying to their pavements;
2. Condition levels at which they would apply different treatments;
3. Treatment information including application cost, surface preparation cost, and life extension provided by the treatments; and
4. An analysis to determine the most cost-effective treatment for each pavement type and condition level.

Once these were determined, a set of decision trees was established for assigning the network-level planning treatments to each management unit needing maintenance or rehabilitation. The actual development of treatments, costs, and decision trees is described by Smith *(6)* and Darter et al. *(8)*. Once the budget needs are determined without considering funding constraints, they are compared with available funds and management units are selected for funding that provide the best return for the money expended. This paper is primarily intended to describe the ranking procedure and analysis used in this selection.

Budget Analysis Concepts

The participating pilot agency public works personnel defined several budget analysis goals they wanted in a PMS. These included

1. The desire to provide the best return for the funds expended,
2. The need to identify funds for capital improvement expenditures separate from maintenance funds,

3. The desire to allocate funds to preventive maintenance as well as rehabilitation,
4. The need to identify deferred maintenance and rehabilitation funds, and
5. The need to consider stopgap or emergency maintenance and requirements.

These requirements were carefully considered in the light of other constraints, especially the need to keep the concepts as simple as possible and minimize the data that must be collected to complete the analysis.

When limited funds must be allocated among a number of different projects, some method of identifying the projects that are considered the most important must be developed. A simple ranking procedure could be used; however, that type of procedure is limited in the number of factors that can be considered. It also generally ranks those in the worst condition as the highest priority without regard to the return on the funds expended. As shown in the economic analysis described by Darter et al. *(8)*, the cost-effectiveness of maintenance and rehabilitation treatments changes with PCI, pavement type, traffic level, and so on. The pilot agencies requested a technique that would consider this but not require complex concepts nor be difficult to understand and use.

Cost-benefit analyses have been adopted as a decision support tool in the transportation field by some agencies *(9, 10)*. Many of the public works supervisors and personnel are professional engineers who are familiar with the concepts included in engineering economics needed for this approach. With all the costs and benefits known, they can be compared directly. Those projects that provide the greatest benefit for the funds expended are then selected *(10, 11)*. However, the benefit in analysis of public financed projects is not simple to define or calculate and if done improperly can be misleading *(12)*.

The initial direct costs to the public agency for pavement maintenance and rehabilitation can be relatively accurately determined; although future maintenance and rehabilitation costs are less well defined, they still can be reasonably estimated. The benefits of pavement improvements are normally based on the concept of reduction in time costs, vehicle operating costs, and accident costs *(10)*. When the facility is improved by decreasing these costs, the resulting savings are defined as user benefits *(13)*.

Considerable effort has been made in the last several years to determine the user benefits associated with travel time and operating costs *(9, 13–17)*; however, they are not always directly applicable to local agency situations and most of the indirect benefits have yet to be determined. To include user benefits in analysis of city and county pavement maintenance and rehabilitation improvements, three components are required:

1. A reasonable set of models that can be used to determine the change in vehicle operating costs due to the maintenance and rehabilitation applied to city and county roads and streets for vehicles traveling at city speeds,
2. A good set of traffic data for use in these models (current models require types and weights of vehicles), and
3. A reasonable method to measure the impact of maintenance and rehabilitation on city streets and county roads that can be related to user benefits (current models require roughness).

The user cost models currently available are based on traffic operating on pavements with 50 to 60 mph speed limits *(16)*, and they appear to be more reliable for determining user benefits related to geometric and capacity improvements than to pavement maintenance and rehabilitation. Fewer than half of the Bay Area cities and counties even have average daily traffic data on most of their streets, let alone traffic data by vehicle class or weight *(5)*. At present, accurate roughness measurements on city streets are expensive to collect, and most Bay Area agencies do not routinely collect the data nor do they have the funds to spend on the measurements.

In general, it is known that as traffic congestion increases and the pavement surface deteriorates, the travel time, vehicle operating costs, and accident costs increase. When maintenance and rehabilitation are applied, there is a period of increased travel time and increased accident occurrence resulting in decreased user benefits or increased user costs. When the improvement is completed, the travel time, vehicle operating costs, and accident rates generally decrease, resulting in increased user benefits *(10)*. Improvements may also allow an increase in traffic, which can affect the price of goods, employment opportunities, property values, and aesthetics. The environment may be adversely affected by construction and the additional traffic that would increase user costs. However, this has not been well quantified for city and country road and street conditions.

Early work on vehicle operation costs are found in works by Sawhill *(14)* and Winfrey *(9)*, and the AASHO Red Book *(15)*. McFarland *(13)* was the first to approximately quantify the effects of the pavement surface in terms of serviceability or roughness on user costs, including vehicle speed, user delays, operating costs, and accident costs. More recent work has been completed by the Federal Highway Administration *(16)* and the World Bank *(17)*, in which costs and benefits were developed as functions of pavement surface condition, highway geometry, and vehicle characteristics.

Of these, only the pavement surface condition, which is primarily measured by roughness in these models, would be affected by the maintenance and rehabilitation managed in the pavement management system of interest. Even then, it would only be the roughness of the pavement surface. In cities and counties, roughness is often caused by drainage structures such as valley gutters, inlets, and other structures that would not be corrected by most road and street maintenance and rehabilitation projects.

The difficulties encountered in determining user benefits have caused many agencies to ignore the user benefits in their analysis *(10)*. Others have used some value that is more easily determined as a surrogate for the user benefits in a cost-effectiveness analysis. The basic concepts of cost-effectiveness are similar to benefit-cost analysis *(12)*.

There are a series of steps required for cost-effectiveness analysis *(12)*. In the first step the goal of the system must be defined. In pavement maintenance and rehabilitation, that goal is to provide the best overall pavement condition for the funds expended. The second step includes the development of alternatives. When selecting pavement management units for funding, a number of different alternative strategies and fundable management units are available. The evaluation criteria must

be selected in the third step, which must provide some measure of the effectiveness of the alternatives. In pavement maintenance and rehabilitation, this can include a measure of the pavement condition and how that condition varies over time, which should be considered over the same time period as the subject improvement. The cost must be calculated in the fourth step, and is usually formulated in terms of life-cycle costs, which consider all costs associated with the system over the life cycle of the alternative. They can be expressed as present worth or they can be annualized. In pavements, they are often divided by some area (lane-mile, square yard, and so on) to normalize for the varying sizes of the pavement management units being considered. The fifth step includes the selection of the fixed cost or fixed effectiveness approach. When multiple alternatives are considered using a ratio of effectiveness to cost, the fixed-cost approach is being used. In the sixth step, the candidate alternatives are ranked in order of their ability to satisfy the selection criteria. This type of procedure can be used to select an alternative for a single project or to select a set of projects to be funded from a group of candidate projects *(11, 18)*.

Pavement alternatives with the same condition but with different levels of traffic do not provide the same benefit. The benefits of the pavement used by the higher-trafficked pavement are greater than those of the lower-trafficked pavement. Weighting can be used to make the effectiveness a function of traffic use as well as condition.

Cost-Effectiveness Analysis Used

Others have used the concept of the area under the performance curve as a surrogate *(18–20)*. Pavements in good condition have lower user costs than pavements in poor condition. In addition, the longer the pavement remains in good condition, the longer it provides lower user costs. The basic hypothesis is that user utility (non-costed benefit) is the mirror image of performance *(19)*. The area above the curve indicates loss of user benefits and the area under the curve is the pavement user utility or non-costed benefit. This is illustrated in Figure 1. The PCI can be used for this curve and is already available in the Bay Area PMS, which eliminates the need for an expensive second set of data collection, roughness.

Effectiveness, then, is defined in the Bay Area PMS as the area under the PCI time curve above the minimum analysis condition level, which was identified to be 25 for the Bay Area PMS. This is illustrated as the area under the curve above the PCI level of 25, shown in Figure 1. The PCI of 25 was determined to be the point below which the Bay Area engineers would generally perform major rehabilitation; the degree of condition below 25 has little impact on the rehabilitation treatment to be applied *(7)*.

Effectiveness is calculated automatically in the Bay Area–PMS software budget analysis module using a trapezoidal integration procedure. For the current pavement surface, the area under the PCI time curve adjusted for performance is calculated for each individual management unit of pavement identified as needing maintenance or rehabilitation in the budget-needs module. The effectiveness of the maintenance or rehabilitation is calculated using the PCI time curve for the individual section adjusted for maintenance or rehabilitation applied at the date it is identified in the budget-needs module. This is illustrated in Figure 2 as the area under the second curve designated as A_3. The first curve represents the adjusted performance curve of the original pavement. The second represents the increase in the PCI to 100 due to an overlay or reconstruction and the projected performance of that pavement following the application of the treatment.

In the above cases, the treatment is applied when the PCI equals 25, and the effectiveness is the total area under the family curve of the treatment applied, shown in Figure 2. However, if a treatment is applied before the PCI of the original pavement reaches 25, the effectiveness of the treatment is equal to the area under the family curve for the treatment minus the area under the existing family curve from the date of

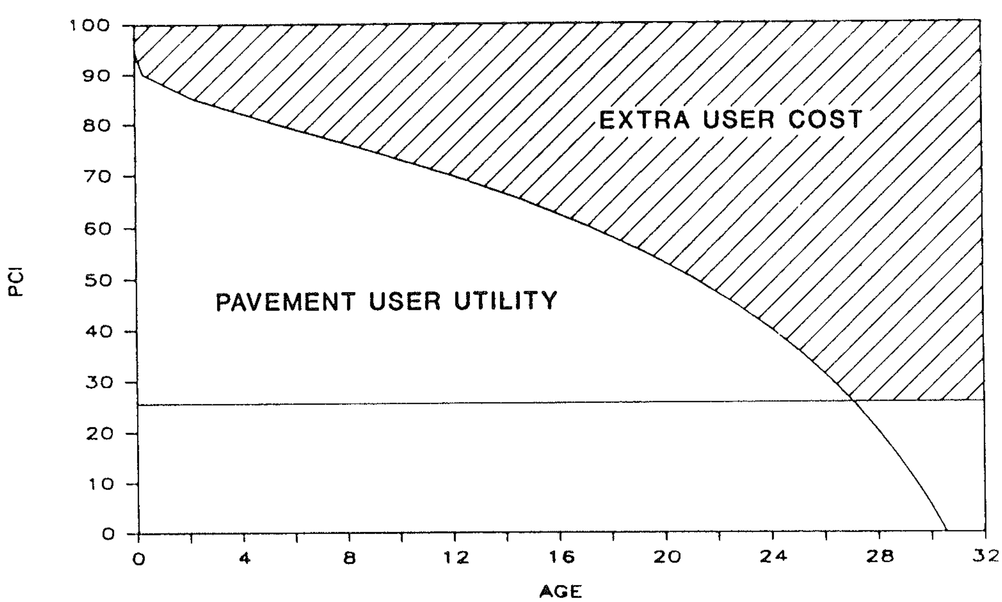

FIGURE 1 Effectiveness equal to pavement user utility shown as the area under the PCI deterioration curve and used in the Bay Area PMS *(22)*.

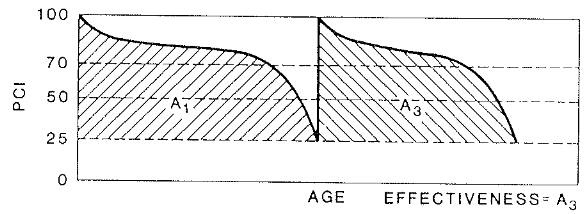

FIGURE 2 Effectiveness of rehabilitation applied at the end of acceptable pavement life shown as A_3.

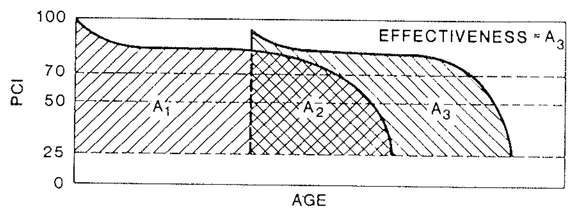

FIGURE 3 Effectiveness of rehabilitation applied before end of acceptable pavement life shown as A_3.

application to the time when the PCI is projected to reach 25. This is illustrated in Figure 3 with the effectiveness of the rehabilitation designated as A_3. The area designated as A_2 is a part of the effectiveness of the current pavement surface and is not a part of the effectiveness of the treatment.

As described below, the costs are annualized. In the effectiveness analysis, the effectiveness is divided by the number of years in the life of the treatment needed to annualize the effectiveness as well.

Cost Calculation

The costs used in the cost-effectiveness analysis are the unit costs determined from the decision trees in the budget-needs module described by Smith *(6)* and Darter *(8)*. To account for the time value of money, the equivalent uniform annual costs (EUAC) are used in the analysis. Both interest and inflation can be used in the equation; however, the user can decide whether only interest or both will be used. To account for the difference in the areas among management units that are being compared, the costs are calculated per square yard.

The effectiveness is an area under the PCI performance curve that is not influenced by the size of the management unit being analyzed. If total cost were used, the management units with the smaller areas would have larger ratios than management units with small areas for the same cost per square yard and same life. Dividing the EUAC by the area normalizes the EUAC for the area. Life extensions of the selected treatments are stored in the data base, with the unit costs for the treatment as default data for use by the PMS software in this calculation of EUAC/yd^2. They may be changed by the user.

Weighting Factors

The ratio of the expected effectiveness per year for the identified maintenance and rehabilitation treatment to the equivalent uniform annual cost/yd^2 (effectiveness/yr)/(EUAC/yd^2) is calculated for each management unit. The management units could then be ranked from highest to lowest cost-effectiveness ratio. The highest to lowest would then be selected until the available budget was expended. However, generally it is less costly to repair the residential or local streets, and in general they will have longer lives than arterial streets. This would cause the majority of available funds to be allocated to residential or local streets. To counter this problem, the effectiveness ratio would have to be weighted for usage. This weighting is normally a function of traffic. When traffic data are not available, as is the case for most agencies in the Bay Area, the weighting could be based on functional classification as a surrogate for traffic.

Multiple attribute decision-making concepts have been developed to determine the importance of various decision-making attributes when used in a forced choice situation *(21)*. A special set of linear programming procedures, termed goal programming, is used for normative decision making. To determine reasonable weighting factors based on functional classification, a set of pavement management units was described in terms of PCI and functional classification. The pilot agency public works personnel were then required to identify the management unit they would repair first if funds were limited to repairing only one of the two. This provided a set of paired comparisons that were used in the goal-programming procedures to develop the relative importance of the ranking compared with the PCI. The weights could then be multiplied by the scale of the attribute to determine the importance. This has been used experimentally by highway researchers to develop relationships between pavements with several attributes *(22)*. However, it is assumed that the scale of the attribute is known.

The PCI scale is well defined; however, the functional classification scale is not known. In fact, if the location of each classification on a scale were known, the relative importance and weighting needed for the effectiveness would be known. The results of the forced choices are shown in Table 1. Analysis of those results provides the ranking table shown in Table 2, where the number in the box indicates the priority, with the lowest number indicating first priority. By trying different locations on a scale of 0 to 10 for the functional classification and the fixed PCI scale, the priority table in Table 2 was very nearly duplicated from results of the goal-programming techniques, as shown in Table 3, by using arterials as 10, collectors, as 7.25, and residential/locals as 5.5. When normalized, this provides a weighting of 1.0 for arterials, 0.725 for collectors and 0.55 for residential/locals. These were selected as the default weighting factors for the PMS.

The user has the option of changing the weighting factors when using the budget-scenario PMS software. The budget-scenario reports can be run using the PMS software for each functional classification separately. The results can then be checked against the results of all the reports using all types combined to determine if the weighting factors are reasonable. This weighting could vary considerably among different agencies based on the traffic levels for the functional classifications and the distribution of functional classifications maintained by the agency. This results in the following equation, which is used to calculate the weighted effectiveness ratio:

$$\mathrm{WER} = \frac{\mathrm{(AREA/YR)\ WF}}{\mathrm{EUAC/SY}}$$

where

WER = weighted effectiveness ratio,

TABLE 1 RESULTS OF FORCED-CHOICE DECISIONS

FUNCTIONAL CLASSIFICATION	PCI	TOTAL CHOICE		FUNCTIONAL CLASSIFICATION	PCI	TOTAL CHOICE
ARTERIAL	10-0	6	VS	COLLECTOR	10-0	0
COLLECTOR	10-0	5	VS	RESIDENTIAL	10-0	1
ARTERIAL	25-10	4	VS	COLLECTOR	10-0	2
COLLECTOR	25-10	3	VS	RESIDENTIAL	10-0	3
ARTERIAL	40-25	1	VS	COLLECTOR	10-0	5
ARTERIAL	40-25	4	VS	COLLECTOR	25-10	2
COLLECTOR	40-25	1	VS	RESIDENTIAL	10-0	5
COLLECTOR	40-25	4	VS	RESIDENTIAL	25-10	2
ARTERIAL	55-40	1	VS	COLLECTOR	10-0	6
ARTERIAL	55-40	1	VS	COLLECTOR	25-10	5
ARTERIAL	55-40	4	VS	COLLECTOR	40-25	2
COLLECTOR	55-40	1	VS	RESIDENTIAL	10-0	5
COLLECTOR	55-40	1	VS	RESIDENTIAL	25-10	5
COLLECTOR	55-40	4	VS	RESIDENTIAL	40-25	2
ARTERIAL	70-55	0	VS	COLLECTOR	10-0	6
ARTERIAL	70-55	0	VS	COLLECTOR	25-10	6
ARTERIAL	70-55	2	VS	COLLECTOR	40-25	4
ARTERIAL	70-55	3	VS	COLLECTOR	55-40	3
COLLECTOR	70-55	1	VS	RESIDENTIAL	10-0	5
COLLECTOR	70-55	2	VS	RESIDENTIAL	25-10	4
COLLECTOR	70-55	0	VS	RESIDENTIAL	40-25	6
COLLECTOR	70-55	5	VS	RESIDENTIAL	55-40	1

AREA = area under PCI curve described above,
YR = years affected,
WF = weighting factor for usage, described earlier,
EUAC = equivalent uniform annual cost, and
SY = square yards in management unit.

Budget Allocation

The pavement management units identified for rehabilitation are separated from those identified for preventive maintenance to determine an appropriate split in preventive maintenance versus rehabilitation by the PMS software. Those identified for rehabilitation are ranked from highest- to lowest-weighted effectiveness/cost ratio within the rehabilitation group, as illustrated in Table 4. A second ranking by weighted effectiveness ratio is completed for the management units needing preventive maintenance, as illustrated in Table 5.

The manager selects a budget for the first year of the analysis period (e.g., $100,000) and an expected budget inflation factor for the 5-year analysis period (e.g., 5 percent). The budget inflation factor is applied only to the budget entered. Any budget inflation factor, including 0, can be selected. The manager also identifies the percentage of the budget to allocate to rehabilitation compared with preventive maintenance (e.g., 70 percent rehabilitation and 30 percent preventive maintenance). These are entered when requested by the menu-driven PMS software.

TABLE 2 PRIORITY TABLE BASED ON ANALYSIS OF FORCED-CHOICE DECISION IN TABLE 34

PCI	ART	COL	RES/LOC
70 - 55	10	13	15
55 - 40	7	11	14
40 - 25	4	8	12
25 - 10	2	5	9
10 - 0	1	3	6

Note: PCI = pavement condition index, ART = arterial, COL = collector, RES/LOC = residential/local.

TABLE 3 PRIORITY TABLE FROM GOAL PROGRAMMING

PCI	ART	COL	RES/LOC
70 - 55	10	14	15
55 - 40	7	11	13
40 - 25	4	8	12
25 - 10	2	5	9
10 - 0	1	3	6

Note: PCI = pavement condition index, ART = arterial, COL = collector, RES/LOC = residential/local.

TABLE 4 MANAGEMENT UNITS NEEDING REHABILITATION RANKING BY WEIGHTED EFFECTIVENESS RATIO

STREET ID	SECTION ID	RH / PM	WEIGHTED EFFECTIVENESS RATIO
ASTREE	02	REHAB	503.96
DEANST	02	REHAB	317.63
ALICES	01	REHAB	310.91
CLAIRE	01	REHAB	308.99
OPTIMI	AREA01	REHAB	293.15
MYRTLE	01	REHAB	230.62
CSTREE	02	REHAB	72.63
CSTREE	01	REHAB	71.77
BSTREE	01	REHAB	70.27
ATHERT	01	REHAB	69.95
WILLIS	01	REHAB	44.78
MYRTLE	03	REHAB	44.78
SUTROS	01	REHAB	44.77
PAMELA	01	REHAB	44.77
FILBER	01	REHAB	44.77
MONTGO	01	REHAB	44.77
DEANST	01	REHAB	39.38

Note: RH = rehabilitation, PM = preventive maintenance.

TABLE 5 MANAGEMENT UNITS NEEDING PREVENTIVE MAINTENANCE RANKED BY WEIGHTED EFFECTIVENESS RATIO

STREET ID	SECTION ID	RH / PM	WEIGHTED EFFECTIVENESS RATIO
DSTREE	01	P. MAINT	702.23
MEEKST	01	P. MAINT	674.14
ALICES	03	P. MAINT	665.39
BURBAN	01	P. MAINT	585.96
ARNOLD	01	P. MAINT	572.40
DOTSON	01	P. MAINT	475.34

Note: RH - rehabilitation, PM = preventive maintenance.

The computer software selects projects identified for rehabilitation from highest-weighted effectiveness ratio to lowest until the funds allocated for rehabilitation (e.g., $70,000) are expended. If the entire rehabilitation budget is not expended, the remainder is allocated to preventive maintenance within the same year; it is not reallocated to following years. Those management units identified as needing rehabilitation but not selected are considered in the following year. Deferred rehabilitation costs are based on the needs deferred in that year. Those management units identified as needing rehabilitation but not selected for funding at this point will also have stopgap maintenance fund requirements assessed based on condition level.

This stopgap maintenance fund requirement is based on the concept that those pavements needing rehabilitation will generate maintenance expenditures to patch potholes and other high-severity and safety-related distresses if they are not rehabilitated. The actual amount allocated to stopgap maintenance is based on an analysis of the type of maintenance Bay Area public works personnel would apply to pavement types in each condition level if funds were not available to apply needed maintenance and rehabilitation. The stopgap maintenance funds assessed by this procedure are subtracted from the funds allocated to preventive maintenance. If the stopgap maintenance fund requirements exceed the available preventive maintenance (PM) funds, then all PM funds are exhausted on stopgap needs. The PMS software reports are then used to advise the user that stopgap fund requirements exceed the allocated funding for that analysis year, and no management units are selected for preventive maintenance. In addition, the surplus PM funds will have a negative balance. Those management units identified as requiring rehabilitation but not selected for the initial year are considered in the following years of the analysis period.

The management units identified to receive preventive maintenance will then be selected based on the same type of ordered weighted effectiveness/cost ratio analysis. Those with the highest ratios are selected until the total allocated preventive maintenance funds for that year, minus those expended for stopgap maintenance, are allocated (e.g., $30,000 − $10,000 = $20,000). Those identified as needing preventive maintenance but not selected in the desired year are considered in the following year of the analysis period.

The total budget allocated to rehabilitation, stopgap maintenance, and preventive maintenance is calculated along with the deferred maintenance and rehabilitation costs and the surplus preventive maintenance funds. This process is repeated for each of the 5 years in the analysis period by the software. The results are provided both in a detailed management unit selection analysis for each year and in a summary table, as shown in Tables 6 and 7. This allows an analysis of the capital improvement budget and preventive maintenance budget compared with other classes.

TABLE 6 DETAILED MANAGEMENT UNIT SELECTION ANALYSIS

Sections Selected for M&R in 1988				
Type of M&R	Street ID	Section ID	Total Cost ($)	Rating
Rehabilitation	FILBER	02	8,358	74.57
	WILLIS	01	19,049	43.69
	MYRTLE	03	29,549	43.69
Total			56,956	
Preventive maintenance	MEEKST	01	10,524	768.22
	DSTREE	01	3,058	650.89
Total			13,582	

NOTE: Budget = $72,223 and 0% inflation, 90% rehabilitation.

The manager can try other budget splits until the best overall network condition is found. In theory, the split that provides the best overall network condition over the analysis period should be chosen. In reality, other factors often intervene. These include the requirement to keep agency forces gainfully employed, limitations on contractor capabilities, and political considerations. In addition, some agencies are constrained to distribute maintenance and rehabilitation funding equally among the political subdivisions of the city or county.

This provides the manager with an analysis tool with which he can look at the effects of the various budget decisions. In effect it is a higher-level ranking approach *(23)*. The ranking has a number of steps and uses more information than most ranking systems. Trade-offs between maintenance and rehabilitation are considered in building the decision trees. It is used again in the ranking process by comparing the effects of various percentages of funds applied to maintenance and rehabilitation. However, an optimization procedure is not included in the PMS software.

There are a number of optimization tools available that could be used to determine the optional allocation of funds *(12, 23, 24)*. However, several factors have inhibited the use of true optimization tools in the Bay Area PMS. First, several of the participating pilot agency personnel adamantly opposed the use of linear programming, Markov decision analysis, and dynamic programming. They felt these techniques were too complex and provided answers through a process they did not understand. On the other hand, the Bay Area PMS system provides a ranking system based on condition of the pavement over time, cost over time, and importance of the road or street in a procedure they understand, support, and can explain to the elected officials to whom they must justify their budget requests.

Second, most of the decisions are made based on the bare minimum of possible data. Ranking provides feasible solutions with improved decision effectiveness; optimization selects the

TABLE 7 SUMMARY DATA FROM BUDGET SCENARIOS REPORT

Type of M&R	Budget Allocation by Year ($) 1987	1988	1989	1990	1991	Total
Rehabilitation	52,826	56,956	60,068	59,063	58,599	287,512
Preventive maintenance	0	13,582	10,069	8,634	11,288	43,573
Stopgap maintenance	18,312	0	1,618	1,911	0	21,841
Deferred maintenance	393,107	350,619	379,802	441,981	402,860	1,968,369
Surplus preventive maintenance	1,085	1,685	468	2,615	2,336	8,189

NOTE: Budget = $72,223 and 0% inflation, 90% rehabilitation.

best; however, when the data are incomplete, optimization may provide no better solution than ranking. The efforts involved in optimization really should be spent optimizing solutions based on more complete data.

Finally, the output that comes from the ranking or optimization at the network level will not be used directly. The output provides lists of management units and treatments. These treatments are basically network-level budget planning treatments developed for budgeting purposes that must be reviewed in later project-level analysis. The maintenance and rehabilitation needs of a small network are shown in Figure 4. Note how the treatment numbers are scattered across the network with the rehabilitation year similarly randomly distributed. If this were the complete network, the agency might apply the treatments that way. However, if this were a small portion of a large network, the difficulty of applying treatments scattered across the network in this way can be imagined. To gain efficiencies of scale, an agency will normally apply the same type of treatment to several streets in an area. They will not apply an overlay to two blocks, heater scarify overlay one block, skip two blocks, apply a chip seal to one block, and skip two more blocks before reconstructing three blocks in the same year. If two management units need a treatment in one year and the management unit connecting them is identified as needing the same treatment the following year, the agency will generally apply the treatment to all three in one year. Thus, considerable modification in management unit selection occurs in developing "contract or maintenance packages."

With this much readjustment of management-unit selection occurring at the project-level following the network-level analysis, it is doubtful that true optimization would provide much better final information than ranking until procedures can be developed that will provide contract or maintenance packages from the PMS. More detailed project-level data are required to develop those packages that cost more money to collect than the agencies are willing to expend at the network level. When the data become available over time, the optimization can be added to the system. Additional budget levels or budget inflation rates can also be tried to determine the impact of budget levels on the overall network condition.

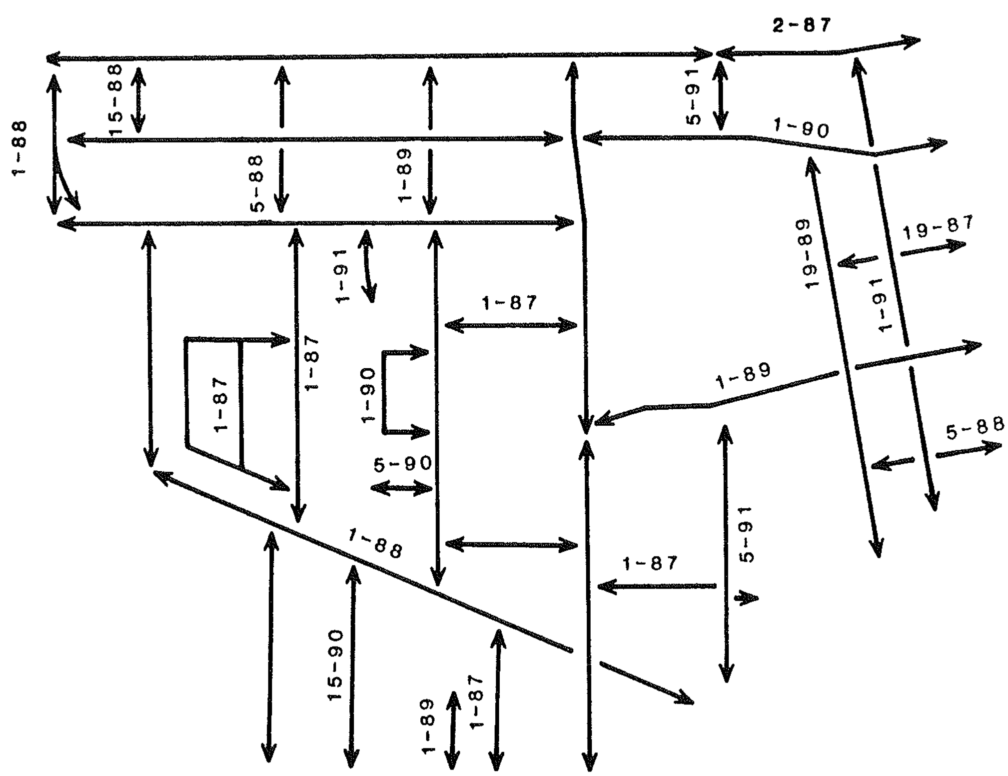

FIGURE 4 Example network with treatment number and year of application (1–86) from PMS budget scenario output.

TABLE 8 PROJECTED FUTURE CONDITION

STREET ID	SECTION ID	LATEST PCI	M & R YEAR	87	88	89	90	91
ASTREE	02	67	87	76	74	72	69	67
DEANST	02	67	87	76	74	72	70	68
ALICES	01	65	87	74	72	70	68	66
CLAIRE	01	63	87	73	71	69	67	64
OPTIMI	AREA01	64	87	73	71	69	67	65
MYRTLE	01	60	87	70	69	67	65	64
CSTREE	02	51	87	100	93	91	90	89
FILBER	02	55	88	52	100	93	91	90
WILLIS	01	22	88	17	100	94	92	91
MYRTLE	03	25	88	20	100	94	92	91
MEEKST	01	76	88	75	83	82	80	79
DSTREE	01	79	88	77	85	83	82	80
ATHERT	01	43	89	40	37	100	93	91
ARNOLD	01	78	89	76	74	84	82	80
DOTSON	01	89	89	88	87	93	92	91
BURBAN	01	85	89	84	83	90	89	88
PARKST	01	40	90	36	32	28	100	94
PAMELA	01	23	90	18	13	7	100	94
ALICES	03	77	90	76	74	73	84	83
BSTREE	02	73	90	71	70	68	77	75
SUTROS	01	17	91	13	8	4	0	100
MONTGO	01	15	91	10	4	0	0	100
ALICES	02	57	0	55	52	50	48	45
CSTREE	01	45	0	42	39	36	33	30
BSTREE	01	42	0	39	36	33	29	26
MYRTLE	02	37	0	34	30	27	23	19
FILBER	01	27	0	22	17	12	6	0
DEANST	01	28	0	25	23	20	16	12
SOUZAC	01	97	91	94	92	91	89	93
WATKIN	01	98	91	94	92	91	89	93
ASTREE	01	97	0	89	86	84	82	80
GRANDS	01	74	0	72	70	67	65	62
GRANDS	02	74	0	72	69	67	64	62
	Network Mean	57.9		58.6	63.0	63.1	66.5	70.7

Note: Budget = $72,223 and 0% inflation, 90% rehabilitation.

Future Network Condition

The future overall network condition is affected by funding available as well as by the allocation of funds to preventive maintenance versus rehabilitation. Individual management unit conditions are projected into the future, reflecting the performance expected with no maintenance or rehabilitation until they are selected for preventive maintenance or rehabilitation. At the time they are selected, the increase in condition because of maintenance or rehabilitation is reflected in the PCI, and the condition of those management units is projected into the future based on the maintenance or rehabilitation applied. Stopgap maintenance does not generally change the PCI significantly nor does it generally provide a long-term increase; it is not reflected in the PCI increases because of maintenance. A table of management unit condition listings with the mean condition of the group for each year is provided to show these results, as illustrated in Table 8.

SUMMARY

The budget-analysis concepts selected for use included providing the best network pavement condition for the available funds. This gives the user the greatest relative advantage. A cost-effectiveness procedure was adopted to provide a relative ranking of pavement management units. This offers observable and readily understandable criteria for ranking that decrease complexity and increase compatibility. The area under the PCI-time curve can be easily visualized and presented to public officials to illustrate the quantity used in selecting management units for funding. Weighting the effectiveness ratio for use increases compatibility with expected results and also increases credibility.

The use of stopgap maintenance in the cost analysis makes the results more realistic in terms of actual maintenance expenditures. The replacement cost procedures make the funds invested in the pavement network more readily apparent to the funding agency personnel. The programs were structured to decrease complexity while the reports were developed to enhance the impact of the presented data to the decision makers.

Treatments, costs, life extensions, and strategies are all default but modifiable elements of the PMS software, and were developed for the Bay Area PMS. These are applicable only for the San Francisco Bay Area, and even then represent the mean costs provided by the pilot agencies. They are used to provide an example of how to develop and use the data, and are not presented as the final answer.

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Expert Systems as a Part of Pavement Management

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Expert systems have recently excited a great deal of interest in all areas of engineering. The availability of affordable software running on mini- and microcomputers has allowed this type of decision tool to move from academe to practical use. Expert systems are discussed in this paper in relation to their place in pavement management systems. The structures of these systems are defined and compared with current pavement management systems. The present state of expert systems is then reviewed. Areas in which pavement management systems can be enhanced are examined, as are the current limitations of these systems.

With the rapid increase in capability and decrease in price of mini- and microcomputers in recent years, fields once thought of as solely the province of university computer science departments have become less esoteric. A striking example is artificial intelligence, which has as a goal making computers act intelligently. Within the field of artificial intelligence, there are several related areas of study including: robotics, machine vision, machine translation, speech synthesis, game theory, and expert systems. Study and development of these areas has expanded from academe to business, resulting in rapid advances. Of all the branches of artificial intelligence, expert systems have produced a great deal of excitement and some of the most concrete results. Because of success with expert systems in other fields, interest is developing in incorporating expert systems' concepts into pavement management systems (PMSs).

The purpose of this paper is to briefly describe the history and development of expert systems, define their current state, examine some long-term research goals, and investigate the usefulness of expert systems' applications to PMSs.

HISTORY

Expert systems research was begun in the late 1950s as an attempt to automate the thought processes of scientists *(1, 2)*. Expert systems were originally built from scratch for each application usually using LISP, the most common programming language for artificial intelligence. Like most computer programs, these early systems mixed rules and data for making decisions with the problem solving process. Such an approach has several drawbacks, which are present in current PMSs and will be discussed later in this paper.

The early expert systems were followed by a landmark program called MYCIN which is still in use. MYCIN was developed by Feigenbaum and Shortliffe to assist doctors in diagnosing bacteriological diseases. MYCIN represented two major advances in the development of expert systems: *(a)* it was the first expert system able to explain why decisions were made, and *(b)* it was the first to separate the data and rules for the decision-making process from the process itself *(2)*. Both of the characteristics are important reasons for incorporating expert system techniques into PMSs.

As expert systems evolved, it became apparent that the process by which decisions were made was somewhat independent of the type of expert system. Although the type of data dictates to some degree how they are manipulated, researchers found that processes for evaluating rules could be used with other data sets for different expert systems. As a result of these observations, the data and rule set were stripped from MYCIN to form EMYCIN, which was labeled an expert system "shell" *(2–4)*. A shell can then be used to create a different expert system without the time and expense required to create a completely new inference engine.

Even though development of the concept of shells was an important evolutionary step, expert systems were still confined to mainframe applications, and were therefore beyond the reach of everyone except universities and very large corporations. A great deal of effort has been expended to adapt expert systems to mini- and even microcomputers, for which relatively sophisticated expert system development tools are now available. Just as the evolution from tailor-made single-purpose expert systems to shells made expert systems more universally accessible, the transition from mainframe to minicomputer is causing them to be adapted for use in all areas of engineering.

EXPERT SYSTEM COMPONENTS

Expert systems are composed of major and separate parts, a knowledge base, and an inference engine. In addition, there are facilities for creation and maintenance of the knowledge base.

Knowledge Base

The knowledge base is made up of data and rules by which conclusions are reached. The data set may be any type of data describing a system, such as background information, historical records, and results of tests. Rules may be laws, mathematical proofs, heuristics, gut feelings, or common sense. Rules are the standards against which the data are manipulated in order to draw conclusions about a problem. There are a number of useful structures for rules, but the most common in practice is

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the IF-THEN-ELSE statement. This type of rule measures whether a condition exists (IF); if it does, one action is taken (THEN); otherwise an alternative action results (ELSE). An example of such a rule is as follows:

IF (given level of pavement distress),
THEN (suggested maintenance or rehabilitation),
ELSE (check for other distress types).

Rules are meant to duplicate the knowledge that a human expert brings to the problem-solving process.

Inference Engine

An inference engine is a collection of processing procedures for examining data-using rules. In one sense, the form of the processing procedures defines the structure of the rules. It can also be said that the type of data and rules determine which type of inference engine is appropriate. There are a number of possible structures for decision making, including the IF-THEN-ELSE statements previously discussed, Markov chains, multidimensional decision trees, and knowledge frames. Each of these structures has its place depending on the situation.

The inference engine is also defined by the way in which the reasoning process flows. Early inference engines used a scheme of production rules *(1)*. The program determined IF-THEN rules in the knowledge base for which there was sufficient information to satisfy the conditional part of the rule. Those rules with satisfied IF statements, were then "fired." The results of fired rules were then checked to see if the goal was met. If the THEN portion of a fired rule matched the goal, the problem was solved. If none of the new information provided by fired rules matched the goal, the facts known by the program, including this new information, were then reviewed to see if additional rules could be fired. This type of reasoning is known as forward chaining or bottom up reasoning *(2)*. The flow of the process is from the low-level information up to the goal.

The process used in MYCIN was backward chaining or top down reasoning. With this type of flow, the program begins with a hypothesis, and determines if any of the THEN portions of rules match. Those that do are triggered, resulting in new information from the IF portion of the rules. The process is completed when the program either reaches facts that are a part of the data base or has to ask for more information. Backward chaining became the standard structure of subsequent expert systems, but many expert system development tools currently available allow the system developer to choose between the two methods. A third alternative is to use some combination of forward and backward chaining. With this method, the tree of information known to the program grows from both the original information and the hypothesis, hopefully meeting to prove the hypothesis.

Another aspect of the inference engine is its ability to handle uncertainty. This facility is especially important for pavement management, where data are not always easily measured and may be collected over a period of many years. Not only is there uncertainty about the correctness of the data, but many of the relationships between different parts of the pavement structure and its relationship to the environment have not been precisely quantified. As a result, there may be a great deal of uncertainty within rules. There are several ways to account for uncertainty in the decision-making process including probability, fuzzy sets, and schemes developed especially for expert systems *(5)*. The developers of MYCIN found that probability was not a concept intuitively grasped by system users, and developed a method of assigning a number from −1 to 1 for the certainty placed on additional information provided by the user.

As previously stated, MYCIN was the first expert system with the ability to explain its reasoning. In conjunction with MYCIN, a program called TEIRESIAS provided a limited ability to answer users' questions on why a certain line of reasoning was followed *(1, 2)*. Many of the expert system shells available for microcomputers have this facility at least in a rudimentary form. This ability may be as limited, as in showing users which rules were invoked, and the paths taken. The value of this feature should not be underestimated; one of the most difficult tasks of computer programming is debugging programs for errors in logic. A means of following the flow of programs is essential for ensuring that the rules drive the reasoning process as intended. Some of the available shells also allow the system developer to add a bit of explanatory text to rules as they are entered *(3)*. In response to a query from the user, the program then displays the text.

Rule Maintenance

In addition to the knowledge base and inference engine, expert systems shells have facilities to build and maintain knowledge bases. At present these include some type of word processing interface to allow the system developer to add and change rules. Although this part of expert systems development has generally received the least attention, it is the most difficult to achieve. Eliciting rules from experts is difficult because often experts don't really know how they solve problems. In many cases they have not tried to quantify the steps to a decision. Another difficulty is that experts often disagree on the causes of problems and acceptable solutions. Creating rules from divergent positions requires a very experienced system developer.

There must also be a means of querying the expert system and receiving the results. A great deal of artificial intelligence research is directed toward developing natural language interfaces. The outcome of this research would be a program that could make sense of a request made in plain English (or any other language), and respond accordingly. Much of the software being offered today claims to have natural language facilities, but there is little evidence to support those claims. Expert system development tools have not advanced to a stage at which the average person with the need for a system will have the resources to devote to develop one. As is the case with PMSs, agencies with a use for such a system will generally look outside for development.

Current Research

There are a number of enhancements to expert systems that are currently under development. One is frames, groups of interrelated rules and data *(6)*. Framing allows faster and more efficient information exchange, reduces redundancy and conflicts

of rules or data, and will aid in the eventual development of model-based expert systems.

Another important area of research is rule checking. Ideally, the expert system should locate rules that:

1. Have indefinite conditions or conclusions,
2. Have become obsolete,
3. Are triggered too often,
4. Are never triggered, or
5. Conflict with other rules or data.

The first three cases are based primarily on the common sense and experience of the expert. At present, the sophistication of expert systems is well below what is required to simulate common sense; however, the last two potential problems are manifested much more directly and distinctly. There is some current effort to include these two types of rule checking into the inference engine.

Recent research has also focused on improving the inference engine's explanation capabilities. Software currently available allows explanatory text to accompany rules. This is a somewhat superficial solution to a problem that might be more thoroughly handled by backward chaining. In essence, the inference engine would retrace its steps in the decision process to expose a critical path. The explanation process has many applications, including debugging, rule calibration, and teaching. As microcomputer speed increases and storage capacities are expanded, much more emphasis will be placed on this aspect.

As described earlier in this paper, the ability to handle uncertainty in the knowledge base is an important feature of the inference engine. Though some recent effort has been made to develop this feature, the primary focus of current research is on soliciting information about uncertainty from the user.

PMS as an Expert System

Pavement management is an excellent test bed for expert systems as it can be argued that PMSs in their current states are rudimentary expert systems, much like the precursors to MYCIN. Current PMSs lack a clear division between their inference engine (normally a single decision tree) and the rule base (typically breakpoints for pavement distress severities and extents). PMSs also lack any explicit explanation capability. As PMSs evolve, there is considerable opportunity for advancing expert system research, primarily in the area of rule manipulation. Data requirements have generally been established. In some areas there is a history of data collection, and there is general agreement on how to quantify pavement serviceability and failure *(7)*.

PMSs provide a unique environment for rule-based evolution for three reasons. First, with continued periodic data collection, there will be opportunities to develop rules from the data to replace the heuristics originally supplied by the experts. Second, as the pavement management system works to improve the road network, the goals of the PMS will change. Third, pavement management is a field in which the recognized experts, whose knowledge will originally be incorporated into the knowledge base, have as counterparts local experts whose experience with local climate, traffic, equipment, materials, work rules, and politics is equally important to the development of a comprehensive knowledge base. It has been suggested that the term expert system is a misnomer, that these systems ought to be called knowledge systems instead, to emphasize the importance of information supplied by those other than acknowledged experts *(8)*. In short, a PMS's rule base should never remain static. The unique organization of expert systems will satisfy this requirement and provide a valuable opportunity to improve the system as well.

LONG-TERM RESEARCH

As computer hardware continues to develop in processing speed and storage capabilities, and as artificial intelligence software becomes more standardized and mature, the field of expert systems will experience phenomenal development. This development will enable computers to simulate human intelligence more accurately rather than to simply respond to input with programmed responses. Two noteworthy possibilities with respect to PMS are Model Based Expert Systems and Intelligent Data Bases.

Model Based Expert Systems

Until now the mechanisms with which the inference engine of an expert system has drawn conclusions are simple rules. These rules, most often in the form of IF-THEN-ELSE statements, are nothing more than sets of conditions associated with instructions to be followed or conclusions to be drawn. In essence, the rules direct the investigation of the inference engine but give no insight into why a line of reasoning is followed. A new generation of model-based expert systems will include the reasons for making inferences and deductions in a certain way *(2)*. Mathematical and heuristic models are by no means new to computer science, engineering, or pavement management; however, the development of an inference engine that can use models to process data, yet remain functionally independent from the knowledge, is far from trivial. Just as the organization of a rule-driven inference engine determines the structure of the rules, so must the model-driven inference engine determine the structure of the models. There must therefore be strict definitions for the form and purpose of the models in the knowledge base.

As the rules in a rule-based system become proven, mature, and properly framed, they will no doubt provide assistance in developing models for future systems. But as models are developed, there must be a concurrent effort to develop model-checking capabilities corresponding to rule-checking features now being researched. This is of foremost importance as models are dimensionally far more complex than are rules. The cause-and-effect relationship of a rule is inherent in its structure. The results of applying individual rules to data are generally transparent. The difficulty in proofing rules comes primarily in their interactions with each other. Models on the other hand are composed of rules. They are often based on intuition and common sense, and applying a model to data can result in actions completely opaque to the user. Thus any inconsistencies, conflicts, or omissions in a model because of unwarranted assumptions may result in subtle errors or divergences which may easily go undetected.

Intelligent Data Bases

A possible solution to the problems of rule/model development and checking may be found in an area of research known as intelligent data bases. Intelligent data bases are distinguished from expert systems in that expert systems use rules or models to derive conclusions from data, whereas intelligent data bases attempt to create rules and models from data. Early work in this area has involved pattern matching, dealing primarily with symbolic representation of data. Although this abstract form of correlation shows promise, researchers remain far from producing computer programs that can derive rules or models from data without human assistance. Nevertheless, the structure of expert systems' rules and models and the organization of the knowledge base into frames provide an excellent conceptual environment for the human expert to apply his experience, expectations, and understanding to improve rules and develop new models. Furthermore, while an intelligent data base may be years in coming, the same concepts can begin to be applied to monitoring rules or models supplied by human experts. Monitoring would be in the form of: *(a)* flagging data that are exceptions to rules, *(b)* indicating divergence in the data from conditions predicted by the models, and *(c)* indicating those rules or models that are either triggered more or less often than expected.

CONCLUSIONS

Unquestionably, expert systems will need considerably more development before they can live up to their expectations. This, however, is not to say that they provide no benefits in their current stage of development. In fact, expert systems present several advantages over PMSs as they are currently formulated.

First, the structure of an expert system is well defined without limiting the analytical or theoretical approaches to data reduction. This can also give definition to an as yet unstandardized category of computer programs, PMS. The most beneficial aspect of this structure is in the separation of the inference engine and the knowledge base. Once the mechanics of the inference engine and the structure of rules and data have been determined, the computer code need not be rewritten whenever new rules or data are added. Thus, the burden on the system programmer is relieved and the maintenance of the PMS is placed back in the hands of the experienced engineer. The pavement engineer is most familiar with the data and is responsible for the answers produced by the system. It is therefore appropriate that he be entrusted with the architecture of the system. Furthermore, the flexibility of the knowledge base allows for continual improvements and updates, and provides the local engineer with a means to customize the knowledge base by incorporating his special knowledge of local conditions.

The second advantage of expert systems is that they can directly address uncertainty in the knowledge base. Uncertainty in data can be handled by appropriate rules and input structures. Rule uncertainty is currently being addressed with techniques such as Markov chains. The task of quantifying uncertainty in the data and rules is still the responsibility of the engineer; however, once the uncertainty has been determined, expert systems can support a structured means for analytically or heuristically accounting for uncertainty.

A third important advantage of expert systems is their ability to explain the reasoning employed to reach conclusions. An inference engine should have the ability not only to use the rules and data in the knowledge base to draw conclusions, but also to retrace its path to explain which rules and data were critical to the conclusion. The benefits of explanation are threefold:

1. Erroneous or inconsistent rules are exposed,
2. The user can have more confidence in the answers received from the system, and
3. The system can be used as a learning tool.

Finally, it should be noted that expert systems are only intended to aid the engineer. They are not substitutes for experience and common sense. As stated previously, the most difficult and important part of developing an expert system is soliciting experts' knowledge. There will inevitably be failures when trying to quantify knowledge that is based on many years of experience. Some expert advice is derived entirely from the expert's intuition, which is inherently unquantifiable although still a valuable source of information. Thus, expert systems are at best tools to organize and enhance PMSs that show great future potential. They will never replace an experienced pavement engineer, and should not be touted as the final solution for PMSs.

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MAPCON: A Pavement Evaluation Data Analysis Computer System

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Described in this paper is a computerized pavement data analysis methodology called MAPCON. Pavement condition data are input and results useful for pavement management are produced. The MAPCON system was developed in a research project funded by the Federal Highway Administration. Eight state highway departments were studied to determine their pavement data collection and analysis procedures. New pavement analysis programs were developed, and existing programs were identified, tested, and modified. All of these programs (a total of 18) were incorporated into a microcomputer package that features menu-driven program flow and fully interactive data input. MAPCON guides the user through selection of analysis method, data entry, and analysis. The path taken by MAPCON is determined by the user's answers to questions presented on the screen. The type of data analyzed by MAPCON includes friction and skid, roughness, structural capacity, surface condition, or a combination of the last three. MAPCON is a set of tools useful to pavement management and design engineers. It is available for implementation and use by highway agencies. Because of the wide variety of existing pavement analysis techniques and ever-changing technology, continued support for further research and development of MAPCON is desirable. Pavement technology is also changing constantly and the MAPCON suite of analysis programs has the capability to change with the technology.

Pavement management is an important concept that involves many highway agency functions such as data collection, planning, research, design, construction, maintenance, rehabilitation, and others. Because of the broad implications for improving the efficiency of building and maintaining the highway pavement infrastructure, pavement management is a major area of emphasis in the United States. The objective of the research described in this paper is to provide highway agencies with a set of tools to reduce raw pavement condition data to suitable inputs for pavement management systems.

OBJECTIVE

Raw pavement evaluation data must normally be processed before it is input to a pavement management system (PMS). Data must be checked for accuracy, entered into a data base, analyzed, and summarized before the engineer can use them to list projects in order of priority or make decisions about rehabilitation. Modern transportable computers allow technicians and engineers to take a computer to the field for direct data entry. This paper describes a system of microcomputer programs to aid in converting raw pavement evaluation data to usable input for a PMS.

From 1979 to 1983, the Federal Highway Administration (FHWA), U.S. Department of Transportation, sponsored research with Pennsylvania State University (Penn State), to assemble several computer programs into a suite of programs called Methods for Analyzing Pavement Condition Data (MAPCON). In a followup project, ARE Inc. has further developed and improved the MAPCON system *(1)*. Three computer programs from the Penn State research were included in MAPCON. Twelve new analysis programs were developed by ARE Inc. and added to MAPCON. Three existing programs were identified, tested, and modified for use in the MAPCON system. This gave MAPCON a total of 18 analysis programs. ARE Inc. then integrated all of the programs into a user-friendly package for convenient use on microcomputers.

MAPCON features menu-driven program flow and fully interactive data input. The system is structured for easy addition of new programs. Also, because of its structure, MAPCON will continue to operate on a microcomputer regardless of the number of programs added.

State highway agencies were selected for participation in the project with ARE Inc. to give a broad spectrum of PMS data-collection equipment, procedures, and analysis techniques. Agency representatives received training in operating MAPCON, reviewed the system, and provided advice about how it could be improved to best fit their needs. These ideas are incorporated into the current form of MAPCON.

The MAPCON system contains procedures for the analysis of pavement condition data. The analysis methods are state of the art in pavement evaluation and design-data handling. Almost all of the basic types of pavement evaluation data analysis procedures are included in MAPCON's comprehensive suite of programs. General types of data analyzed by MAPCON include friction or skid, serviceability or roughness, structural capacity, and surface distress. Both rigid and flexible pavements may be considered. The program is user friendly and easy to learn and operate by anyone familiar with pavement evaluation and design concepts. The user does not need special computer expertise, but a working knowledge of MS-DOS (disk operating system) is helpful. MAPCON runs on IBM-PC compatibles using MS-DOS version 2.1 or greater. New pavement analysis programs may be introduced by simply adding them to the menus and providing a simple interactive data input routine (if desired). The analysis routines exist as individual executable files on computer disk. Only one routine is called into computer memory at a time so the maximum size of the entire MAPCON suite is virtually unlimited.

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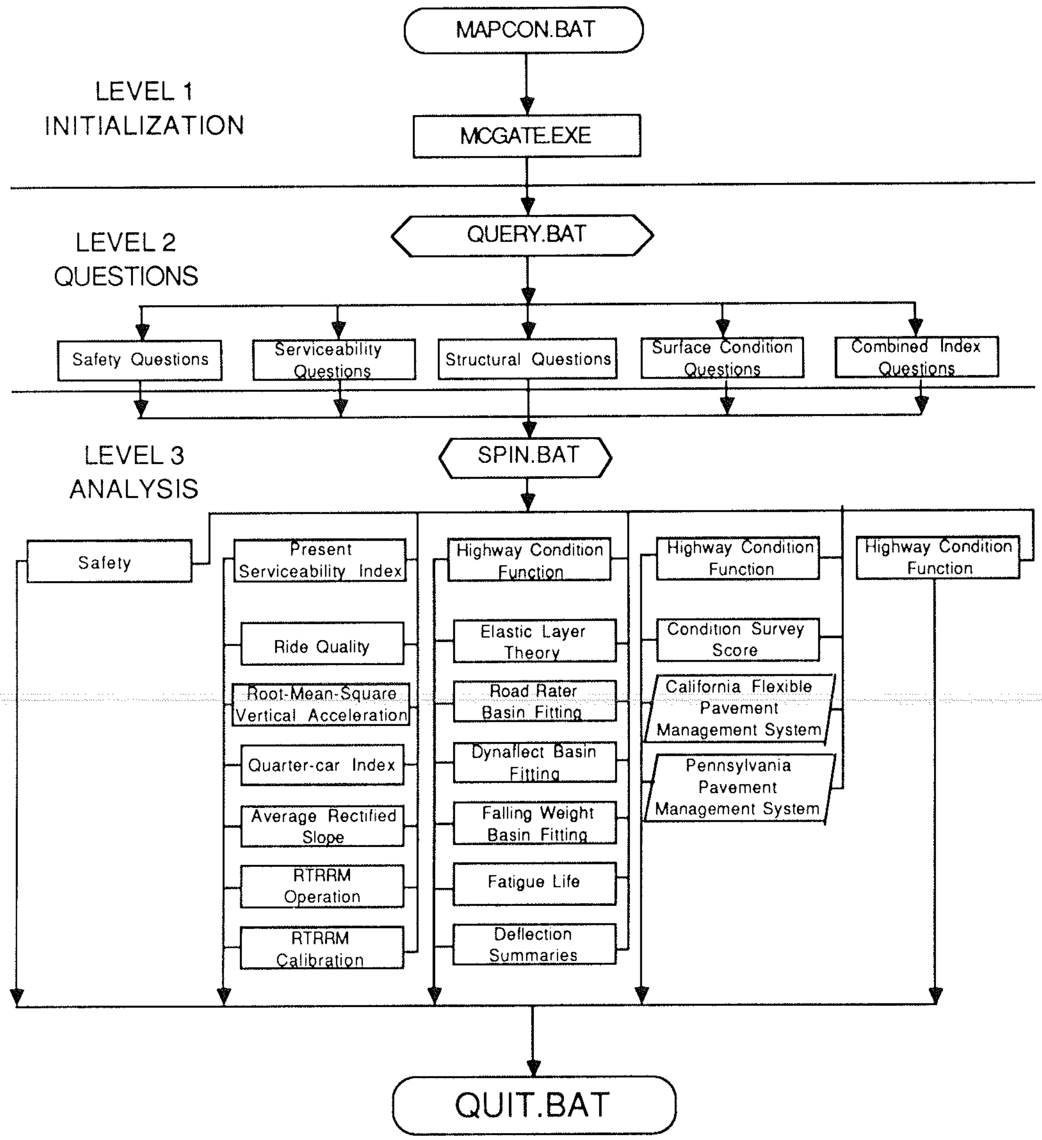

FIGURE 1 Flowchart of overall MAPCON system showing individual program modules.

ANALYSIS PROCEDURES INCLUDED IN MAPCON

MAPCON allows the user to select the desired analysis method, enter data, and perform analyses. Figure 1 is a diagram showing the basic flow of MAPCON. The user initially types the word "MAPCON". From there, the path taken by MAPCON is determined by the user's answers to questions presented on the screen. The initial question concerns the type of pavement data to be analyzed. The choices are

1. Safety,
2. Serviceability or roughness,
3. Structural capacity,
4. Surface condition, or
5. A combination of the last three.

Depending on the answer, MAPCON executes the appropriate path setting data-entry program. The user is presented with menus of options. His choices determine the type of analysis to be performed. The program then presents data-entry questions. Data checking is performed by reprinting the answers and allowing them to be changed. For the input of bulk data, such as pavement profiles or pavement texture data, the user has the option of entering data in the program or inputting the names of existing bulk data files. After data entry is complete, MAPCON executes the chosen analysis programs with the input data files.

A complete description of the operation of MAPCON and the algorithms used are given in the *MAPCON User's Manual* (*2*) and the *MAPCON Operations and Maintenance Manual* (*3*). An overview of the programs and instructions for use are also in the User's Manual.

Pavement Safety Analysis

The safety-analysis routines involve the determination of friction and hydroplaning potential for a standardized tire, and the analysis of skid resistance/velocity relationships from friction measurements. The safety analysis paths in MAPCON are all included in a subprogram called SAFE; the flow is shown in Figure 2.

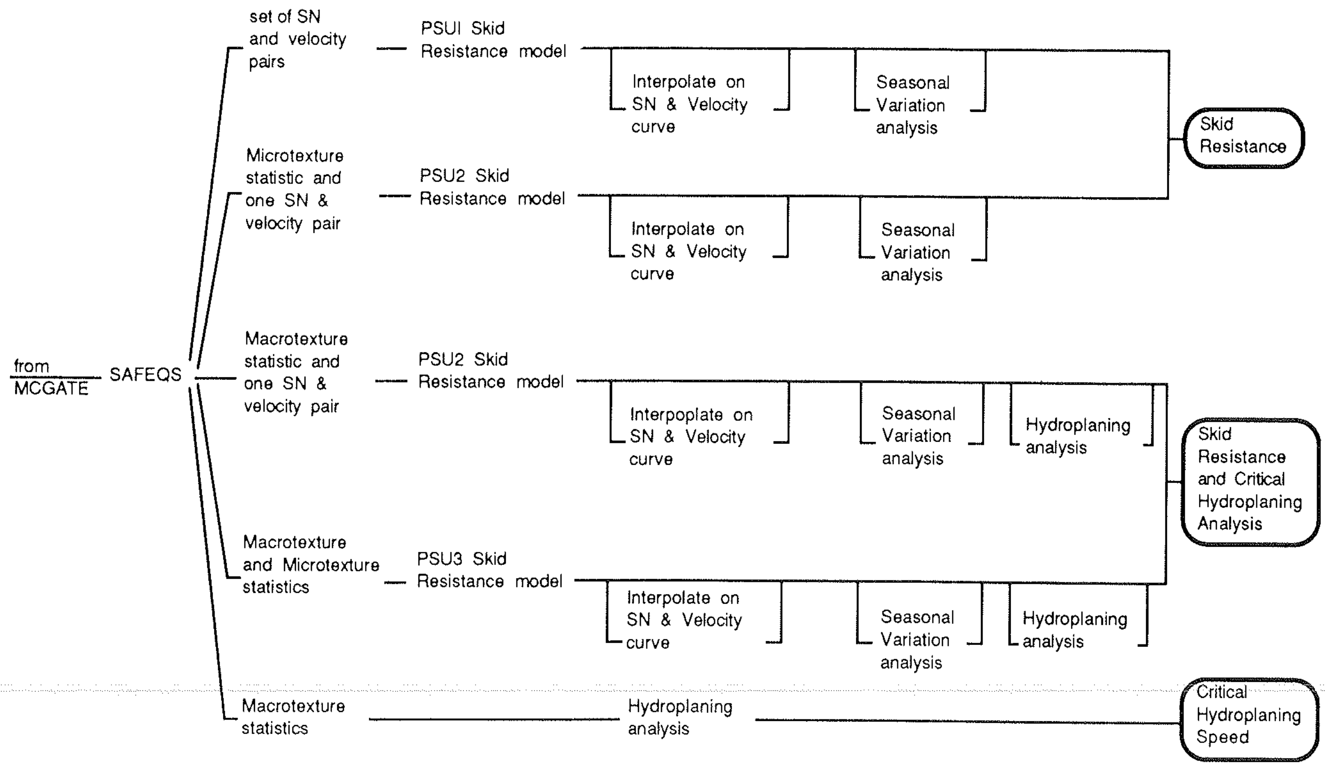

FIGURE 2 Safety analysis paths.

Program SAFE uses combinations of skid number with associated vehicle velocity and microtexture or macrotexture profile data to define a skid number/velocity relationship for a pavement. Routines are also included to calculate critical hydroplaning speed, given a macrotexture parameter.

Pavement Serviceability or Roughness Analysis

The concept of rating the present quality of a road is discussed in detail by Carey and Irick *(4)*. Present serviceability is defined as "the ability of a specific section of pavement to serve high-speed, high-volume, mixed (truck and automobile) traffic in its existing condition." As the term and its definition indicate, the present serviceability index (PSI) relates only to the condition of the pavement at the present time and not to its past or future condition. The present serviceability rating (PSR) is the mean of the subjective evaluations of present serviceability made by a human rating panel. The panel is intended to represent all highway users. The PSR (and therefore the serviceability) of a pavement has been shown to be directly related to pavement roughness, as measured by mechanical equipment, in a number of studies *(5–7)*. The paths available to perform serviceability and roughness analysis within MAPCON are shown in Figure 3.

Several types of data can be used to estimate serviceability. The most common types of data collected are

- Profile data collected by profilometers or by rod and level surveys.
- Roughness data collected using response-type road roughness meters (RTRRMs).

Profile data consist of pavement elevations collected at regular, closely spaced intervals along the traveled path of the road. It is generally not cost effective to collect profile data for an entire pavement network.

Response-type devices such as Mays Meters, Bureau of Public Roads (BPR) roughometers and Portland Cement Association (PCA) meters are cost-effective for collecting data on a network level, but the raw data cannot be used to estimate serviceability directly. RTRRMs measure the response of a vehicle to road roughness. The response for a given section of road will be different for each device used. The response of an RTRRM is therefore machine dependent and must be calibrated.

MAPCON contains several programs to evaluate profile data and calibration and operation data from RTRRMs. These programs are called PSI, RQ, VAARE, QIARE, ARSARE, RUNMM, and CALBMM, and are described as follows.

Program PSI contains routines for analyzing several types of data to get a present serviceability index. Data types include output from several types of RTRRMs, pavement profile data, and axle-body displacement data. Routines include simulations for ideal and conventional Mays Meter; CHLOE profilometer; and quarter-car, half-car, and full-car models. There is also a paver-grinder simulation routine.

Program RQ analyzes several types of data to give ride comfort statistics. One model included is the University of Virginia Ride Quality Model (8), which analyzes pavement profile data and root-mean-square body acceleration and roll rate data. Another model is the ISO Ride Quality Model (9), which uses pavement profile data and either a linear or a nonlinear transfer function.

Three programs use pavement profile data to produce pavement roughness statistics. These statistics can be used to calibrate RTRRMs. Program VAARE calculates the root-mean-

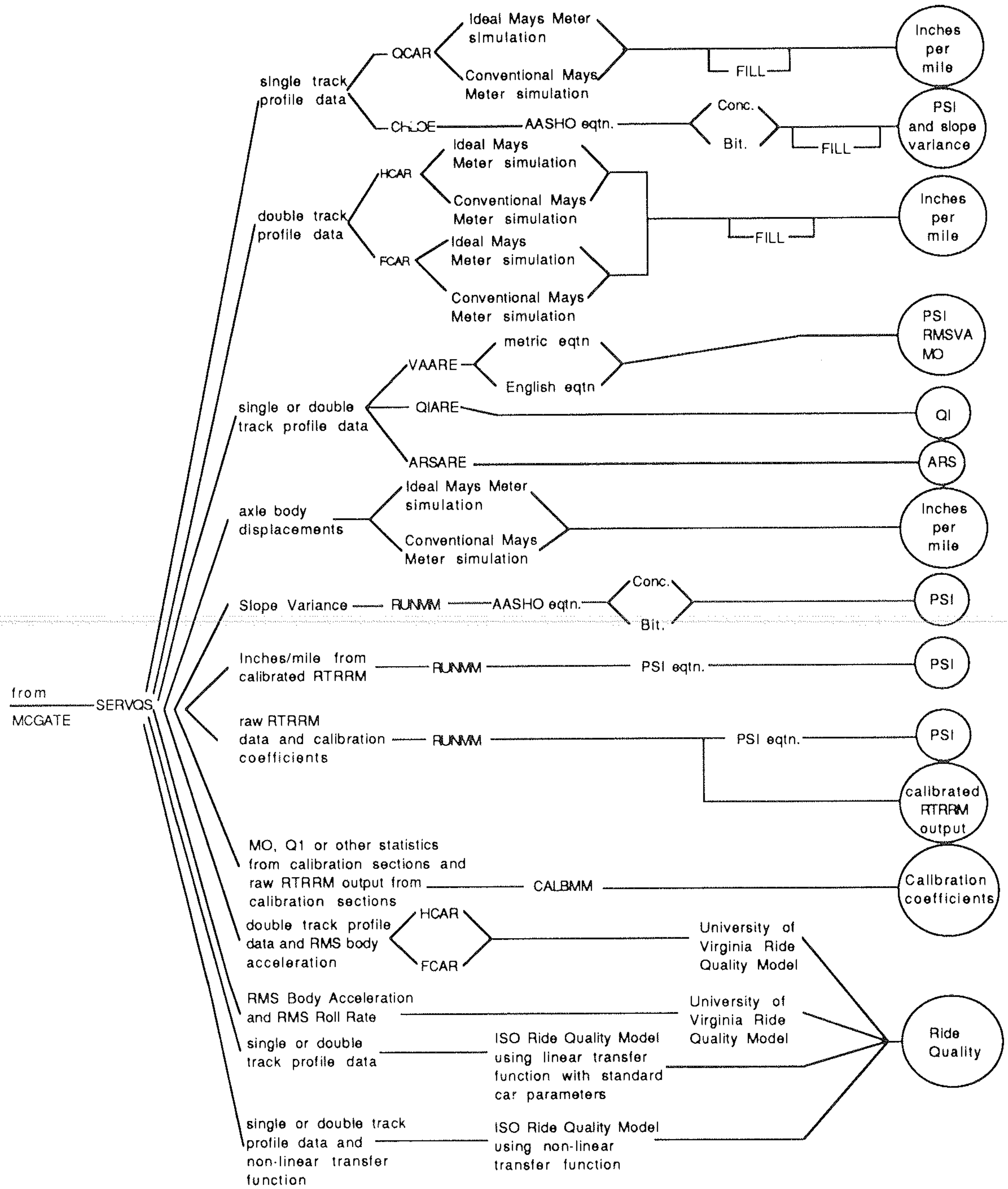

FIGURE 3 Serviceability analysis paths.

square vertical acceleration statistic (RMSVA). Program QIARE calculates a quarter-car index (QI). Program ARSARE calculates average rectified velocity (ARV) and average rectified slope (ARS) as defined by Gillespie, Sayers, and Segel *(10)*.

Programs CALBMM and RUNMM are used to develop RTRRM calibration coefficient files and convert RTRRM data to standard roughness statistics using these files. Calibration coefficient files can be set up using roughness statistics such as RMSVA, QI, ARS, ARV, or other statistics.

Pavement Structural Capacity Analysis

The purpose of structural capacity analysis is to determine the ability of a pavement to withstand loads. The primary types of data used to determine structural capacity are

- Modulus of elasticity values, Poisson's ratios, or other fundamental engineering properties of the pavement materials.
- Deflection data collected using multisensor devices such as the Dynaflect, road rater, or falling weight deflectometer.

MAPCON has several methods of evaluating data from both categories. The available structural analysis paths are shown in Figure 4. The programs used in the structural capacity paths are RRFIT1S, DYNAFIT, FWDUT1S, ELSYM5 (ELSARE), HCF, FATLIF, and GENDEF, which are all described in the following paragraphs.

Programs DYNAFIT, FWDUT1S, and RRFIT1S are used for interactive deflection basin matching for the Dynaflect, falling weight deflectometer, and road rater, respectively. These

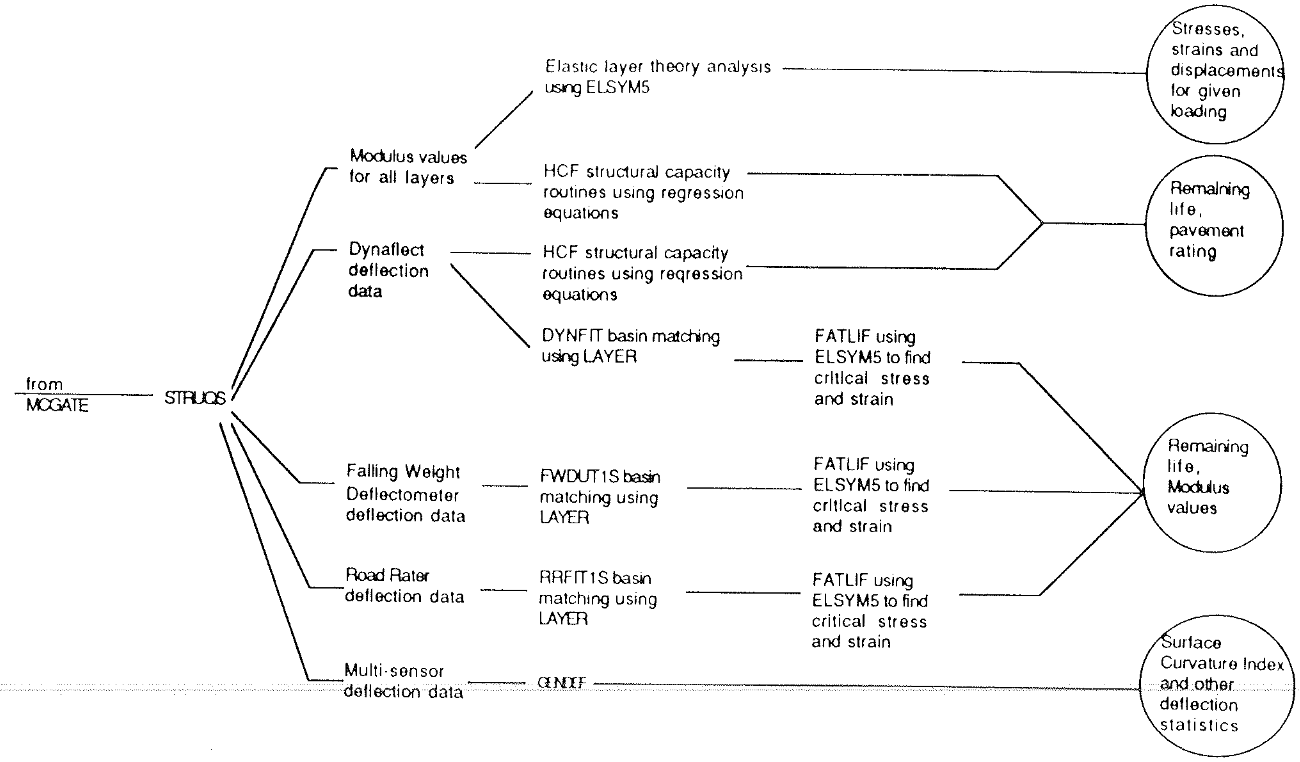

FIGURE 4 Structural capacity analysis paths.

programs use elastic layer theory to estimate a deflection basin. The input material properties are varied until the theoretical basin matches the measured basin. Each program has fully interactive data input and plotting of the calculated and measured deflection basins. They also have the capability to analyze pavement structures with a bottom layer of bedrock.

Program ELSYM5 contains routines for elastic layer theory analysis of a pavement structure. This program uses an input format developed by ARE Inc. It provides estimates of stresses, strains, and deflections at user-specified locations within a pavement system.

Program HCF uses simulated elastic layer theory analysis routines and fatigue-life equations to predict fatigue life from either Dynaflect deflections or input layer modulus values.

Program FATLIF uses the fatigue-life equations previously mentioned to predict pavement fatigue life. It reads input files created by the deflection basin matching programs, or by the user. Elastic-layer theory routines are called to find the critical stresses or strains used in the fatigue-life equations.

Program GENDEF takes any multisensor deflection-device data and calculates surface curvature index (SCI), base curvature index (BCI), and spreadability index (SPR). It calculates the mean and standard deviation of the sensor readings, SCI, BCI, and SPR for all the deflection basins in each section.

Pavement Surface-Condition Analysis

Surface-condition analysis involves the manipulation of data concerning distress manifestations that are present on the pavement surface. The surface-condition analyses included in MAPCON are shown in Figure 5.

Surface-condition data are used in MAPCON for two purposes:

1. To calculate a pavement-condition survey score, or pavement condition index; and
2. To create a data base for use in making PMS decisions.

Four paths are available in MAPCON for these purposes. The programs that make up these paths are HCF, FPMS, STAMPP, and SCORCS, all of which are described as follows.

Program HCF contains routines that calculate the pavement-condition index of the PAVER system *(11, 12)*. The result is a summary report of the distresses with their associated severities and extents and an overall pavement-condition index.

Program FPMS is a computerized pavement management system for flexible pavements developed by the state of California *(13)*. It features full-screen data entry and editing, data file manipulation, and determination of rehabilitation options.

Program STAMPP is also a pavement management system developed by the Pennsylvania Department of Transportation *(14)*. It has full-screen data entry, data file manipulation, and determination of rehabilitation options, and, like FPMS, is written in BASIC.

Program SCORCS calculates a condition survey score based on a user-defined equation. The user decides whether weighted distress deducts are added to or subtracted from a perfect pavement score. The user defines the perfect pavement score and inputs the weighted distress deduct values for each distress. The program then calculates overall pavement scores for use in prioritization of sections.

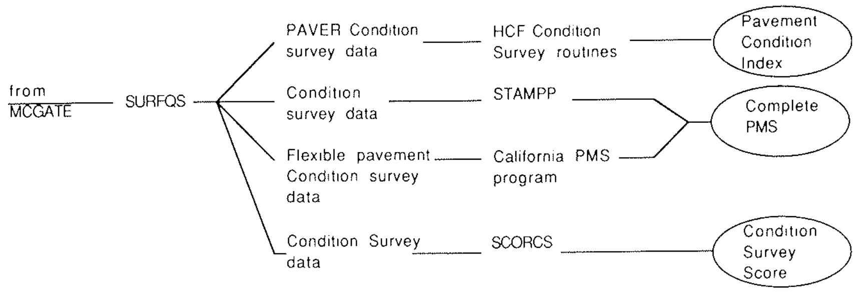

FIGURE 5 Surface-condition analysis paths.

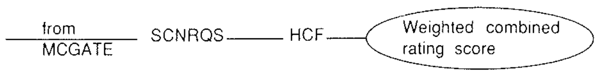

FIGURE 6 Combined structural capacity, condition survey, and ride-quality analysis paths.

Combined Analysis of Pavement Serviceability, Structural Capacity, and Surface Condition

The highway condition function (HCF) as already mentioned, is a computer program for establishing a pavement condition rating function. Figure 6 shows the path for this analysis. Measures of surface distress, structural capacity, and roughness are combined into overall indices that describe the condition of the pavement. The program uses any combination of one, two, or all three of the pavement condition parameters. Three forms of pavement condition summary index are output:

1. Weighted average,
2. Variance on minimum, and
3. Matrix rating value.

A weighting value is assigned to each of the pavement fitness measures and the average is computed for the weighted average statistic. The variance on the minimum uses the statistic with the lowest normalized value (i.e., the fitness measure indicating the worst-pavement condition) and adds the variance between the other pavement fitness measures to define the overall rating of the combined measures. In the matrix rating value approach, each of the pavement fitness measures is divided into six levels. The six levels for the three fitness measures define a $6 \times 6 \times 6$ matrix of pavement condition. The user can define a maintenance or treatment strategy for each of the cells in the matrix. These are used to define a dominant strategy for the treatment of the pavement.

The pavement condition function is modified with traffic and environmental factors to produce an overall highway condition rating score. A regression equation was developed to quantify the opinions of several highway engineers with respect to the weighting factors that should be used for the environmental and traffic factors. The user may readily change the default factors. For a detailed description of the highway condition function model, see Carmichael et al. *(15)*.

CONCLUSION

Summarized in this paper is a description of a computer program developed on a research project entitled "Procedures for the Evaluation of Pavement Condition Data." The research covered current practices in pavement-condition data analysis and incorporated relevant analysis procedures into a suite of computer programs, called MAPCON. This suite contains state-of-the-art procedures for all aspects of raw pavement-condition data analysis to produce inputs for pavement management systems.

The MAPCON suite of programs is useful to highway agencies in several respects. It is an automated method for analysis of raw pavement condition data to produce meaningful inputs to a pavement management system. MAPCON is also a useful tool for training pavement engineers and designers in the data types and methods used in pavement evaluation and rehabilitation. MAPCON is an analysis package to be used by highway agencies to reduce their pavement-condition data to usable indices and statistics.

Whereas each of the individual programs is powerful, the real strength of the MAPCON suite of programs is the ease of use. During the research project, more than 30 engineers from 8 states were trained to use the program. The training sessions generally consisted of one morning of introduction followed by structured use of the program in the afternoon. During the second day of instruction, the participants were experimenting freely with the program. With the ready availability of low-cost and powerful computers, programs such as MAPCON should significantly advance the state of the art in needed analysis tools. The boundaries of technology will continue to be pushed by research. MAPCON helps make new technology available to the highway practitioners.

ACKNOWLEDGMENTS

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A Microcomputer Procedure to Analyze Axle Load Limits and Pavement Damage Responsibility

DAVID R. LUHR AND EMMANUEL G. FERNANDO

The development of rational guidelines for the posting of load limits in Pennsylvania is presented. A theoretical analysis was conducted to evaluate the effect of axle loads under a variety of conditions that considered various load magnitudes and configurations for different pavement thicknesses and material properties. It was found that axle configuration (i.e., single-, tandem-, and triple-axle assemblies) did not significantly affect pavement response, provided that the load per tire remained the same. A performance model based on present serviceability index was developed that related pavement performance to calculated subgrade strain. In order to accommodate Pennsylvania's deflection-measuring equipment, a procedure was developed that determines the subgrade strain from measurements taken with either the road rater or the falling weight deflectometer. A microcomputer program was written that incorporates the new procedure and includes a default traffic stream that is typical of secondary roads. The program generates information concerning predicted years to failure for different load limits. In addition, simple charts were developed to allow engineers to conduct a load-limit analysis in the absence of deflection measurements and to determine pavement damage responsibility for different axle loads. Results of an example application of the procedure indicate more damage responsibility for heavy loads on thin pavements than on thick pavements, as would be expected. However, cost allocation based on marginal pavement damage can be misleading if the initial cost of construction is not considered. The load-limit analysis procedure presented in this paper can be a valuable tool in the evaluation of axle load limits and axle damage responsibility.

Every state has specified the maximum legal load limit for a single axle, for a tandem axle, and for maximum gross vehicle weight (GVW). Often, however, roads do not have adequate structural capacity to carry axle loads at the legal load limit for all or part of the year. In the spring when the ground is thawing, these pavements have significantly reduced bearing capacity. In addition, for many roads the expense of importing non-frost-susceptible materials is prohibitive. To deal with this problem, many legal codes allow for the posting of load limits below the state's legal maximum.

This reduction (posting) of legal load limits on roads during the spring-thaw period is employed in 17 U.S. states and Canadian provinces *(1)*. Some of the states employ deflection-measurement equipment to determine when the road is in its worst condition. The Washington State Department of Transportation (WSDOT) uses load restrictions that are based primarily on experience and, occasionally, on either Benkelman beam or falling weight deflectometer (FWD) surface deflection measurements. Alaska has developed quantitative methods of establishing load restrictions based on measurements of pavement surface deflections. Most states use experience and judgment to determine the appropriate posted load limits for spring thaw conditions.

In Pennsylvania, the use of load-limit posting goes beyond the seasonal load restrictions employed by some states. The reason is that the Commonwealth of Pennsylvania is responsible for 44,000 miles of roadway, approximately two-thirds of which are classified as collector or local roads. In most states such lower volume roadways would be the responsibility of local government agencies. Because many of the secondary roads have a structural capacity that is inadequate to carry heavy loads without extensive maintenance, Pennsylvania has the authority to post load limits during any time of the year. Currently the state has a uniform, load-restriction policy: when load limits are imposed, they are always set at 10 tons GVW.

A major factor in the consideration of axle load limits on a particular roadway is the cost associated with the pavement deterioration caused by each vehicle. For occasional overloads, permits can be purchased. Similarly, if vehicles heavier than the posted limit use the road for an extended period of time, proper bonds may be required. At present, both types of fee are determined on the basis of experience.

The posting of load limits on the basis of GVW poses a fundamental problem. The load from the vehicle is transmitted through the axle tires, and the load applied by each tire depends on the number of tires per axle. Tandem and triple axles have more tires than have single axles, so they can carry a heavier load while putting as much stress on the pavement as a lighter-loaded single axle. Because the performance is related more accurately to axle loads and axle types than to GVW, the posting of load limits should be based on a maximum load for a given axle type.

The objective of this research project was to develop a rational and comprehensive guideline for the posting of load limits. The procedure had to be capable of evaluating the load-carrying capacity of pavements and of determining the appropriate damage cost to be assigned to heavy vehicles. The research effort resulted in the development of a comprehensive procedure for the evaluation of axle load limits. This procedure, which was developed for a microcomputer, uses deflection measurements from either a road rater or an FWD to determine the expected pavement life for different axle load limits. In addition, the program determines the percentage of

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total damage resulting from each axle load, so that estimates of damage responsibility can be made. For cases in which the user does not have deflection-measurement information, simple figures were developed to provide the engineer with results for typical pavement conditions.

REVIEW OF CURRENT PRACTICE

Load-Limit Posting Practices in Pennsylvania

The establishment of truck-axle weight restrictions below the legal load limits is authorized by Section 4902 of the Pennsylvania Motor Vehicle Code. Under this law, Commonwealth and local authorities may impose restrictions on the weight or size of vehicles allowed to operate on a particular route whenever it is determined that, without such restrictions, excessive damage may occur to the road. Section 4902 also authorizes Commonwealth and local authorities to issue permits allowing the movement of vehicles that exceed the limits of size and weight, and to require sufficient security to cover the cost of repairing the pavement damage caused by the movement of heavy vehicles. The procedure for bonding the roadway is usually as follows:

1. The operator who wishes to haul amounts in excess of the posted load limit notifies the state department of transportation;
2. The department inspects the roadway, so that future damage (caused primarily by the operator) will be recognized;
3. The operator posts a bond, indicating the obligation to maintain the road in a suitable condition; and
4. The department periodically inspects the road for operator compliance.

Even though the procedure requires a significant amount of manpower from the department to be implemented, there are some offsetting benefits to the department. For example, the operators will typically contract out the required maintenance work on their own, thereby reducing the amount of maintenance work required by department forces. Also, as each operator is financially responsible for his own road, he is more careful to monitor his truck loads and reduce hauling when the pavement is in a condition with poor bearing capacity.

In Pennsylvania, the maintenance districts enforce a 10-ton gross vehicle weight limit for posted roads. Chapter 15 of the Pennsylvania Department of Transportation (PennDOT) *Maintenance Manual* establishes a uniform, statewide policy on hauling in excess of posted load limits *(2)*. This load-limit specification was selected on the basis of engineering judgment and experience. Most posting in the districts is done on a permanent (year-round) basis, although seasonal posting of some routes is practiced.

Although the establishment of load restrictions on the basis of gross vehicle weight is convenient from the standpoint of implementation, it is fundamentally incorrect. The load from a vehicle is transmitted through the axle tires, and the number of tires per axle significantly affects the loads transmitted to the pavement. Consequently, pavement response is more directly related to the tire loads imposed on the pavement surface than to gross vehicle weight. It is therefore more rational to determine load limits on the basis of axle loads and number of tires per axle.

DEVELOPMENT OF LOAD-LIMIT ANALYSIS PROCEDURE

Analysis of Axle Loads and Configurations

An important objective of this project was the analysis of the effect of axle-load distribution on pavement response. This analysis was conducted by examining theoretical solutions of a linear-elastic, pavement-analysis computer program called BISAR *(3)*. With this program the effect of changing load magnitude or load configuration for a variety of pavement conditions could be evaluated in a practical and rational way.

A three-layer pavement structure (surface, base, and subgrade) was selected for the analysis because it is representative of typical pavements for secondary roads in Pennsylvania. Three different levels (associated with low, medium, and high values) were chosen for various pavement parameters (surface thickness, surface modulus, base thickness, base modulus, and subgrade modulus). Because of the importance of load magnitude in this study, five different levels were selected for this variable. The levels of all variables were selected with equal differences between levels in order to satisfy certain criteria in the statistical analysis of the data. The values chosen for the different factor levels (Table 1) represented a broad range of pavement and loading conditions, and include the range of surface and base thicknesses typically found in Pennsylvania. The possible combinations of all values of all factors (a full factorial) result in 3^5 times 5, or 1,215 observations for each axle configuration. The pavement surface deflections, the horizontal strain at the bottom of the asphalt concrete layer, and the vertical strain at the top of the subgrade were calculated for all of the factorial combinations. These pavement-response variables were determined for single-, tandem-, and triple-axle configurations.

A detailed analysis of the pavement-response study is reported elsewhere *(4, 5)* and is too long to repeat here. It was found that axle configuration (i.e., single-, tandem-, and triple-axle assemblies) did not significantly affect theoretical pavement response, provided that the load per tire remained the same. It was therefore decided to use load per tire as the principal factor in developing a load-limit analysis procedure.

Following the analysis of axle loads, a performance model based on present serviceability index (PSI) was developed that related pavement performance to calculated subgrade strain. The performance is given by

$$\begin{aligned}\log_{10} N_X = {} & 4.508 - 436.992\,(\varepsilon_{sg}) + 0.092\,(H_2 + H_3) \\ & + 0.141\ (\mathrm{PSI}_i * \mathrm{TSI}) - 0.014\,[\mathrm{TSI}(H_1 + H_2 + H_3)] \\ & + 3.382 \log_{10}(H_1 + H_2) - 0.319 \log_{10} \\ & [(\mathrm{PSI}_i * H_2) + 1] - 1.987 \log_{10}(\mathrm{TSI} * H_1) \\ & - 0.299 H_2 - 0.00018 P + 0.041\,(H_1 * H_2)\end{aligned}$$

$$R^2 = 0.758 \quad \mathrm{SEE} = 0.283 \quad N = 568 \text{ observations} \qquad (1)$$

where

N_X = number of applications of axle load X,
ε_{sg} = maximum subgrade vertical strain,
H_1 = surface layer thickness (in.),
H_2 = base layer thickness (in.),

TABLE 1 LEVELS OF VARIABLES USED IN THE STUDY

Variable	Levels	Units
Load (all dual tires)		
(a) Single Axle	6; 12; 18; 24; 30	kips
(b) Tandem Axle	12; 24; 36; 48; 60	kips
(c) Triple Axle	18; 36; 54; 72; 90	kips
Surface Thickness -T1	1; 5.5; 10	inches
Surface Modulus -E1	80×10^3; 540×10^3 1000×10^3	psi
Granular Base Thickness - T2	3; 9; 15	inches
Granular Base Modules - E2	10×10^3; 40×10^3 70×10^3	psi
Subgrade Modules - E3	3×10^3; 10×10^3 17×10^3	psi

H_3 = subbase layer thickness (in.),
PSI_i = initial present serviceability index,
TSI = terminal serviceability index, and
P = load per tire (lb).

In order to accommodate Pennsylvania's deflection-measuring equipment, a procedure was developed that estimates the subgrade strain directly from deflection measurements taken with either the road rater or the FWD. In the development of the procedure, theoretical displacements for the road rater and FWD loading conditions were determined from multilayer linear elastic theory *(4, 6)*. For the road rater, a loading frequency of 25 Hz and a peak-to-peak displacement of 0.1 in. were assumed in the calculation of theoretical surface displacements. These assumed values for frequency and peak-to-peak displacement result in a peak-to-peak force of 500 lb and are those normally used by PennDOT when road rater deflection measurements are taken. Theoretical displacements were determined at four different positions corresponding to the four sensors of the road rater, which are spaced at 1-ft intervals.

For the FWD, a load level of 9,000 lb, applied through a circular plate of 5.9-in. radius, was assumed in the computation of theoretical surface displacements. The displacements were determined at seven different positions, corresponding to the seven sensors of the FWD, assuming a 1-ft spacing between sensors.

The theoretical displacements calculated by BISAR for the road rater and FWD loading conditions, and for each combination of layer moduli and thickness included in the factorial study presented earlier, were subsequently correlated with theoretical strain values associated with various axle loads and axle configurations. The regression equations obtained are given in Table 2.

FRAMEWORK FOR ESTABLISHING LOAD RESTRICTIONS

With the development of the strain versus deflection relationships and the formulation of a strain-based performance model, a rational framework for evaluating load restrictions was developed (see Figure 1). In the load-limit analysis procedure, deflection measurements taken with either the road rater or the FWD are used to estimate subgrade compressive strains caused by various axle loads in the traffic stream. The axle-load distribution provided by the pavement engineer is converted to an equivalent, tire-load distribution, and the number of allowable applications for each tire load present in the traffic stream is determined from the performance model. By going through a procedure in which the cumulative pavement damage is successively calculated as tire loads of increasing magnitude in the traffic stream are considered, a curve such as that shown in Figure 2 can be constructed. The determination of load limits for posting can then be made by specifying a minimum time that a road must remain in service before rehabilitation is allowed.

The curve shown in Figure 2 will vary depending on pavement structural condition and on the characteristics of the traffic stream for the road segment under consideration. In the procedure, pavement structural condition is evaluated from deflection measurements. The traffic distribution must be provided by the pavement engineer. Unfortunately, this information is usually not available or is not collected on a regular basis, particularly for secondary roads. These types of roads are the ones that are most often posted in Pennsylvania. Consequently, efforts were made during the study to define typical traffic distributions for secondary roads, information that can be used in the absence of actual data.

TABLE 2 RELATIONSHIPS FOR ESTIMATING SUBGRADE COMPRESSIVE STRAIN FROM DEFLECTION MEASUREMENTS

$$\log_{10}(\varepsilon_{zz})_{X,FWD} = -4.273 + 0.433 \log_{10}(W_1 - W_2) + 0.560 \log_{10}(W_1 + 2W_2 + 2W_3 + W_4) - 1.799 \log_{10}(H_1 + H_2) + 0.912 \log_{10}(P_{tire})_X + 0.122 \sqrt{H_1} + 0.285 \sqrt{H_2}$$

$R^2 = 0.9715$ SEE = 0.088 N = 3645 obs

$$\log_{10}(\varepsilon_{zz})_{X,RR} = -2.784 + 0.498 \log_{10}(W_1 - W_2) + 0.477 \log_{10}(W_1 + 2W_2 + 2W_3 + W_4) - 0.948 \sqrt{H_1} + H_2 + 0.9121 \log_{10}(P_{tire})_X + 0.097 H_1 + 0.673 \sqrt{H_2}$$

$R^2 = 0.9703$ SEE = 0.090 N = 3645 obs

where

$(\varepsilon_{zz})_{X,FWD}$ = vertical compressive strain at top of subgrade due to tire load X, computed using FWD measured deflections

$(\varepsilon_{zz})_{X,RR}$ = vertical compressive strain at top of subgrade due to tire load X, computed using Road Rater measured deflections

W_i = measured deflection at the i^{th} sensor of the deflection device used, inches

H_1 = surface layer thickness, inches

H_2 = base layer thickness, inches

$(P_{tire})_X$ = tire load, lb

SEE = standard error of estimate

DETERMINATION OF TYPICAL TRAFFIC DISTRIBUTIONS

PennDOT was contacted to determine the availability of data on axle-weight distribution for secondary roads. For these classes of roads, it was found that PennDOT has data on traffic counts broken down by vehicle type but very little information on vehicle axle-weight characteristics. Collection of axle-weight data is mainly done on the primary routes (Interstate and principal arterials), but very little information is gathered on the secondary routes (minor arterials and local roads).

However, W-3 tables for the 1982 to 1984 period for local roads were obtained from PennDOT. These tables were based on survey data from three survey stations located on secondary roads in the state. The tables provide information on average vehicle weights by type of vehicle and for both loaded and empty conditions. Additional information was obtained from a truck-weight case study conducted by FHWA in a year-long study in 1980 to 1981 *(7)*.

Tables provided by FHWA showing the distribution of gross vehicle weights among the various axles were used, together with the data available on average gross vehicle weights, to establish distributions of axle loadings for secondary routes. The axle-load distributions were converted to equivalent tire-load distributions by dividing each axle load by the appropriate number of tires per axle. Figures 3 and 4 show cumulative distributions for tire loads as determined from PennDOT data on local roads. Figure 3 shows the tire-load distribution for loaded vehicles, while Figure 4 shows the distribution when

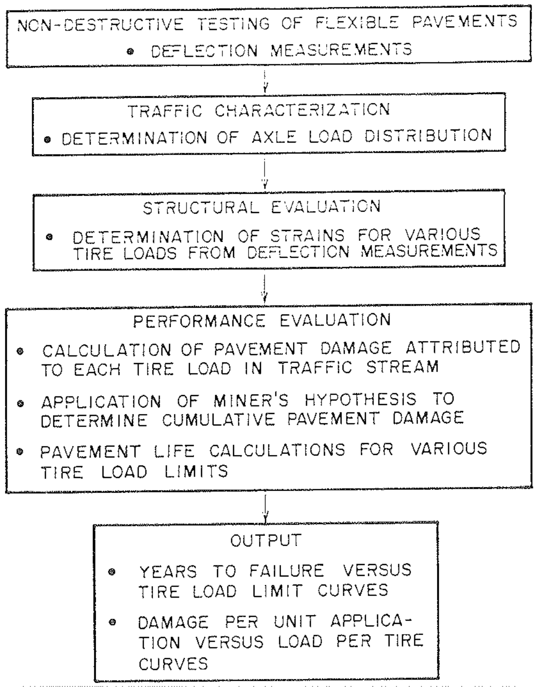


FIGURE 1 Rational framework for evaluating load restrictions.

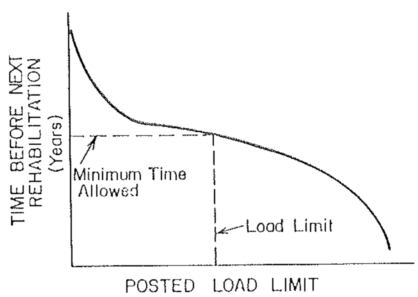


FIGURE 2 Selection of load limit based on minimum time to next rehabilitation.

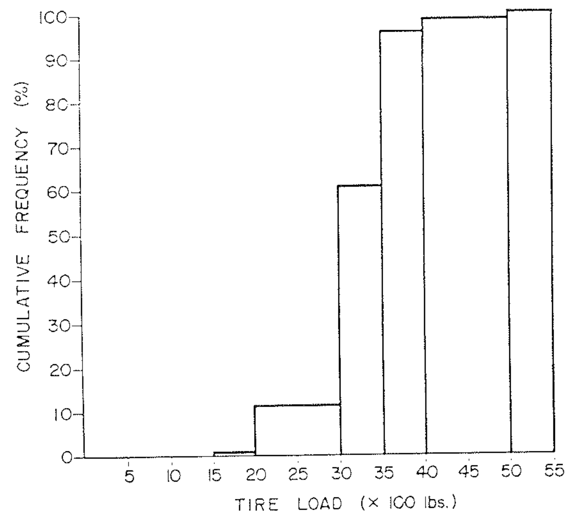


FIGURE 3 Cumulative tire-load distribution (loaded vehicles).

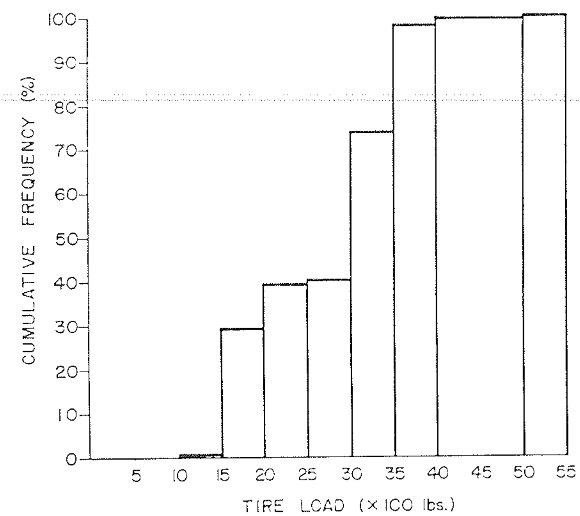


FIGURE 4 Cumulative tire-load distribution (loaded and empty vehicles).

both empty and loaded vehicles are considered. A load-limit analysis of both distributions showed that the results were similar because the predicted pavement performance was dominated by the loaded vehicles—information that appeared in both distributions. In the absence of site-specific traffic survey data, the cumulative, tire-load distribution given in Figure 3 is provided as a default in the load-limit analysis procedure. However, the pavement engineer is cautioned not to use the default load distributions indiscriminately because they may be significantly different from the actual traffic conditions. Actual truck traffic count and weight surveys for characterizing the traffic stream are strongly recommended.

In the load-limit analysis procedure, the given traffic volume, annual average daily traffic (AADT) remains constant as different load limits are considered. For example, in reference to Figure 3, if the AADT is set at 1,000 vehicles/day, and a load limit of 4,000 lb/tire is being considered, the procedure will distribute the 1,000 vehicles/day according to the load-distribution figure for loads less than or equal to 4,000 lb. This is a simplifying assumption that does not consider a shift in the load-distribution curve for different load limits. However, if the user has information concerning the shifted, load-distribution curve, this information can be input directly and used in the procedure.

DEVELOPMENT OF A COMPUTER PROGRAM AND GENERAL CHART FOR LOAD-LIMIT ANALYSIS

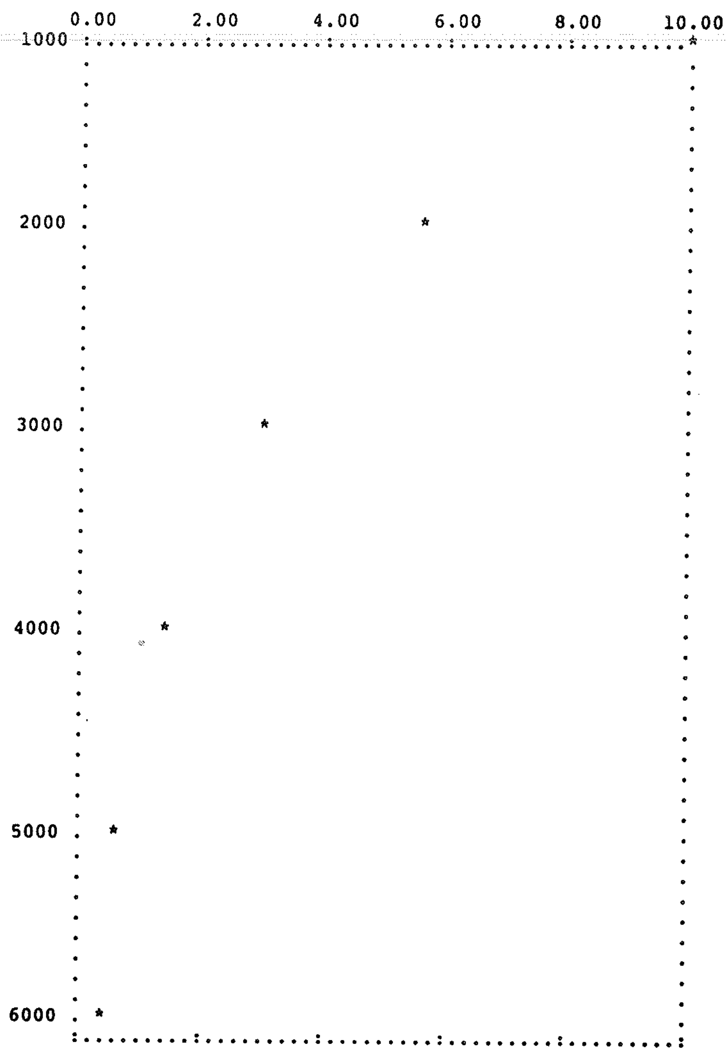
The load-limit analysis procedure discussed here has been implemented in a computer program. The program is interactive and is suitable for use on a microcomputer.

The program requires deflection measurements made with either the road rater or the FWD. Information on tire-load distribution, if available, can also be entered in the program. Otherwise, the default tire-load distribution presented in Figure 3 is used for the load-limit analysis. Output from the program includes a plot of the years to failure versus load per tire curve, and a plot of the inverse of the number of allowable applications before failure ($1/N_f$) versus load/tire. Figures 5, 6, and 7 illustrate sample program output. Figure 6 is used to evaluate

```
DATE  03/24/86

DISTRICT NO.     2                         BEGINNING JOB STATION        0011+00
COUNTY   14                                ENDING JOB STATION    0013+25
LEGISLATIVE ROUTE NO.    L 135             MAINTENANCE CLASS     A
WHEEL PATH    1                            LANE        RIGHT

DEFLECTION DEVICE USED IS ROAD RATER
NUMBER OF DEFLECTION BASINS TAKEN IS  10

      TIRE LOAD DISTRIBUTION
  TIRE LOAD   DAILY APPLICATIONS
     500.00        100.00
    1500.00         70.00
    2500.00         40.00
    3500.00         30.00
    4500.00         28.00
    5000.00         30.00

TOTAL DAILY LOAD APPLICATIONS (BOTH DIRECTIONS) =      298.00000000
BASIN STATION INDEX FOR LOAD LIMIT ANALYSIS      3
BASIN STATION ID    0011+50
LAYER 1 THICKNESS      3.00
LAYER 2 THICKNESS      4.00
CURRENT PSI    3.50
TERMINAL SERVICEABILITY INDEX    1.50
```

FIGURE 5 Sample output from LOADLIM (data on tire-load distribution) entered as inputs.

TIRE LOAD LIMIT (lbs) VS YEARS TO FAILURE 0011+50



SCALE FACTOR OF X: E 0   SCALE FACTOR OF Y: E 0

FIGURE 6 Sample output from LOADLIM: Plot of load per tire versus number of years to failure.

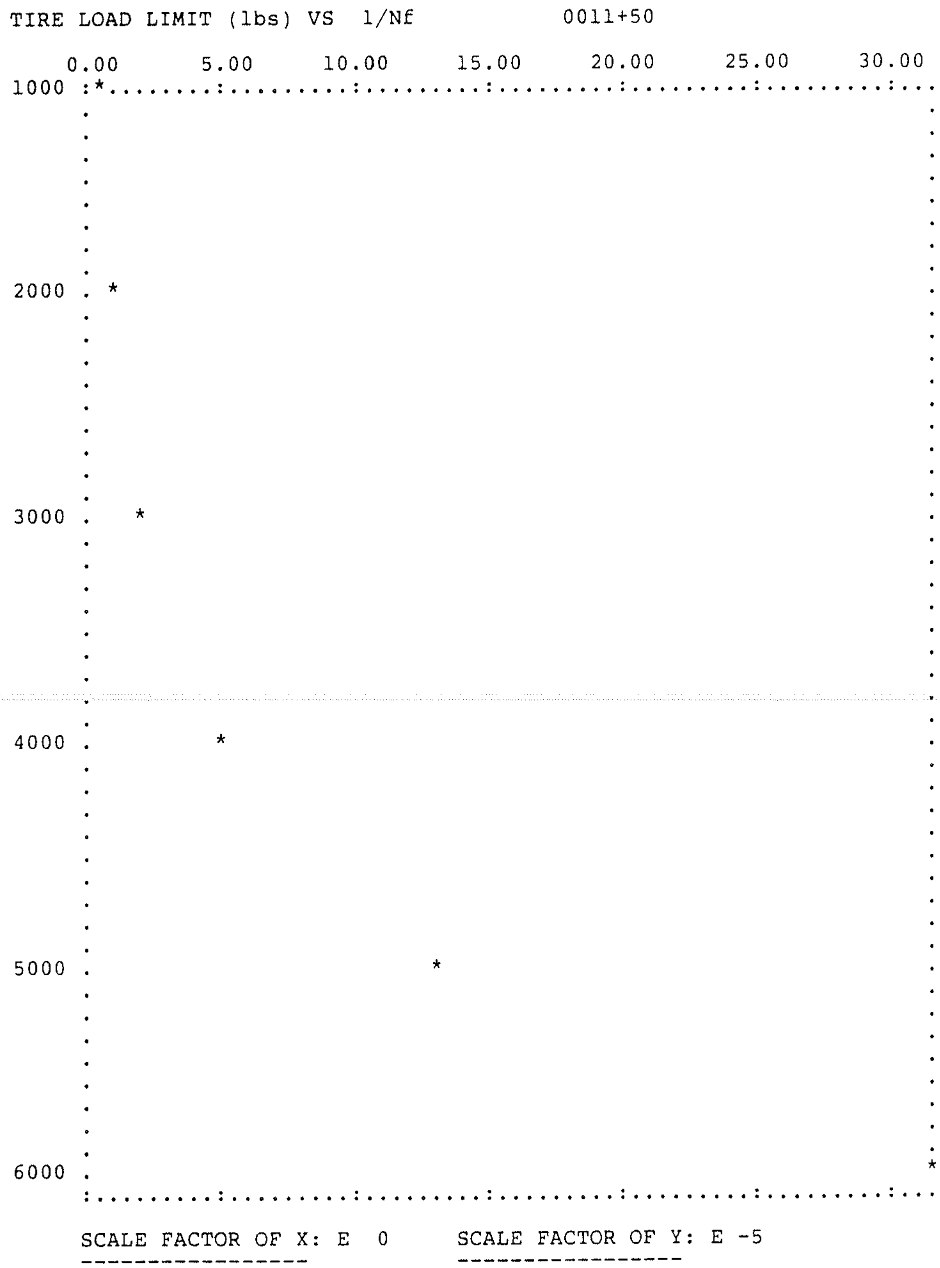


FIGURE 7 Sample output from LOADLIM: Plot of load per tire versus damage per unit load application.

the sensitivity of pavement design life to load limit. As illustrated in Figure 2, the user may have a desired minimum time before rehabilitation and can determine the load limit appropriate for that time to failure. Figure 7 indicates the fraction of total damage caused by one load application of an axle with a given load per tire. This figure can be used to determine damage responsibility for different axle loads and is discussed in more detail in the next section. The program also creates a file called DEF.OUT (Figure 8) containing the deflection measurements entered by the user and a plot of Sensor 1 deflections versus station index. This plot is generated if three or more deflection basin measurements are entered by the user.

Inasmuch as deflection data may not always be available to the pavement engineer, a chart (Figure 9) has been prepared that allows load limits based on a qualitative evaluation of pavement structural condition to be determined. In the development of the chart, the assumed values of larger moduli and thicknesses for medium- and poor-quality pavements were characterized as follows:

	Pavement Condition	
Variable	*Medium*	*Poor*
Asphalt-layer modulus (psi)	300,000	150,000
Base modulus (psi)	35,000	15,000
Subgrade modulus (psi)	6,500	3,000
Surface-layer thickness (in.)	3.5	1.5
Base thickness (in.)	5.0	4.0

The tire-load distribution assumed is that presented in Figure 3.

The general chart and the computer program form a complete package, which allows the pavement engineer to conduct

```
DATE  03/24/86

DISTRICT NO.     2                          BEGINNING JOB STATION       0011+00
COUNTY    14                                ENDING JOB STATION    0013+25
LEGISLATIVE ROUTE NO.    L 135              MAINTENANCE CLASS     A
WHEEL PATH    1                             LANE        RIGHT

DEFLECTION DEVICE USED IS FWD
NUMBER OF DEFLECTION BASINS TAKEN IS  10


          PLOT OF MAXIMUM DEFLECTION VS. STATION NO.

             0.00       1.00       2.00       3.00
            1 :........*:.........:.........:.....
            2 .         *                        .
            3 .      *                           .
            4 .      *                           .
            5 .                  *               .
            6 .                                  *
            7 .          *                       .
            8 .            *                     .
            9 .          *                       .
           10 .          *                       .

    SCALE FACTOR OF X: E  0       SCALE FACTOR OF Y: E -2
    -----------------             ----------------
```

INDEX	STATION ID	DEF #1	DEF #2	DEF #3	DEF #4
		SENSOR DEFLECTION (inches)			
1	0011+00	0.008760	0.006004	0.003327	0.001693
2	0011+25	0.010276	0.007717	0.004705	0.002756
3	0011+50	0.005787	0.004941	0.003622	0.002559
4	0011+75	0.005906	0.005000	0.003740	0.002717
5	0012+00	0.025236	0.016752	0.008031	0.003661
6	0012+25	0.034724	0.020669	0.008189	0.002795
7	0012+50	0.010787	0.007835	0.004744	0.002795
8	0012+75	0.013484	0.009311	0.005236	0.002835
9	0013+00	0.010709	0.007717	0.004724	0.002795
10	0013+25	0.010709	0.007677	0.004764	0.002894

FIGURE 8 Sample listing of DEF.OUT file.

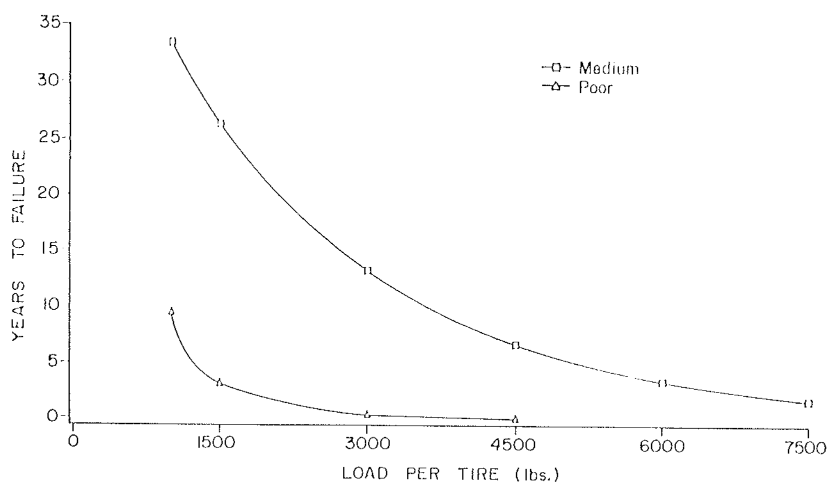


FIGURE 9 Chart for establishing load restrictions based on a qualitative evaluation of pavement structural condition.

a rational evaluation of load restrictions both with and without deflection data. The collection of deflection and traffic data is recommended, however, because the data would permit a better evaluation of the effect of different load-limit policies.

DETERMINING PAVEMENT DAMAGE RESPONSIBILITY FOR DIFFERENT AXLE LOADS

Methodology

For a variety of reasons, it sometimes becomes necessary to determine the amount of pavement damage resulting from a given axle load. This is often difficult because the AASHTO load-equivalence factors are not sensitive to the effect of different pavement structures *(8)*. In this project, a procedure was developed for estimating the amount of damage for which each vehicle is responsible.

The principal factor involved in determining damage responsibility is the expected number of applications to failure for each axle load. As the concept of cumulative damage (Miner's Rule) is used in the performance algorithm, if a pavement is expected to carry 100,000 applications of a light vehicle before failing, then each application of that vehicle consumes 1/100,000 of the pavement life (or 0.001 percent). However, if only 1,000 applications of a heavy vehicle are required to make the same pavement fail, then each heavy vehicle application consumes 1/1,000 (or 0.1 percent) of the pavement life. By multiplying the percentage of pavement damage per vehicle application by the actual number of applications, the total damage caused by that vehicle can be determined.

The percentage of damage responsibility can be converted to cost responsibility by considering the cost of rehabilitating a certain roadway. For example, if the cost of rehabilitating a secondary road is \$50,000/lane mi, and a vehicle is determined to cause 0.0003 percent damage per application, then the cost associated with each application of that vehicle would be

$$\$50{,}000 \times \frac{(0.0003)}{100\%} = \$0.15/\text{mi/vehicle}$$

If the computer program is not used, then Figure 10 can be used to estimate the percentage of damage for an axle with a given load per tire. Curves are provided for two example pavements, one medium-quality and one poor-quality pavement. Details on the assumptions for these pavement examples are given in the preceding section.

Application Examples

To demonstrate the use of the procedure in determining damage responsibility, a load-limit analysis was performed on three sample pavements. The three examples are actual pavement sections that are part of the 1-mi pavement loop of the Pennsylvania Transportation Research facilities at Pennsylvania State University *(9)*. Deflection measurements from the pavement sections were used in the procedure as was also the following thickness information:

	Thickness of Asphalt Concrete (in.)	*Thickness of Granular Base (in.)*
Section 1	5.5	6
Section 2	7.5	6
Section 3	9.5	6

A traffic rate of 2,500 AADT was used in the example, using the default load distribution given in Figure 3.

The results showing the time to failure versus the tire load limit for the three pavement section examples are given in Figure 11. It is interesting to note that the predicted pavement performance for Section 3 is the one affected most by the change in axle load limit. This is due to the fact that Section 1 is probably under-designed for the chosen traffic level and the pavement fails relatively early regardless of the load limit.

Of primary interest in this example is the determination of damage responsibility, as shown in Figure 12. The results

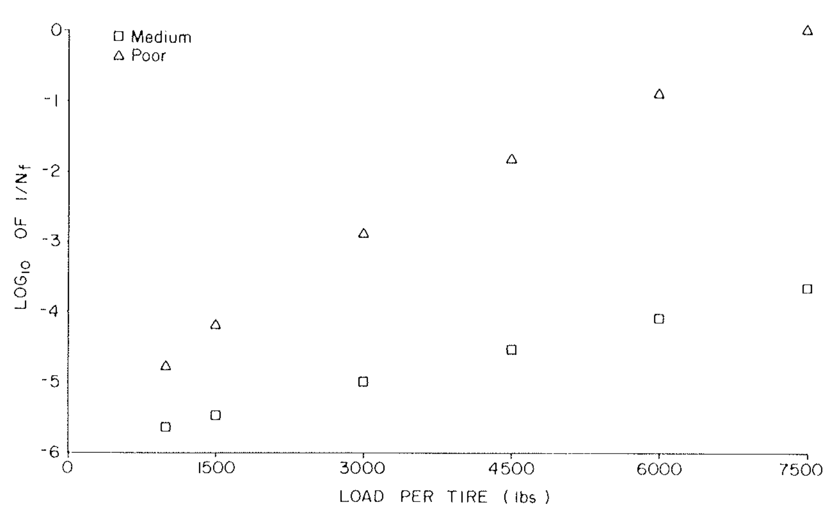


FIGURE 10 Contribution to pavement failure of various tire loads for two example pavement structures.

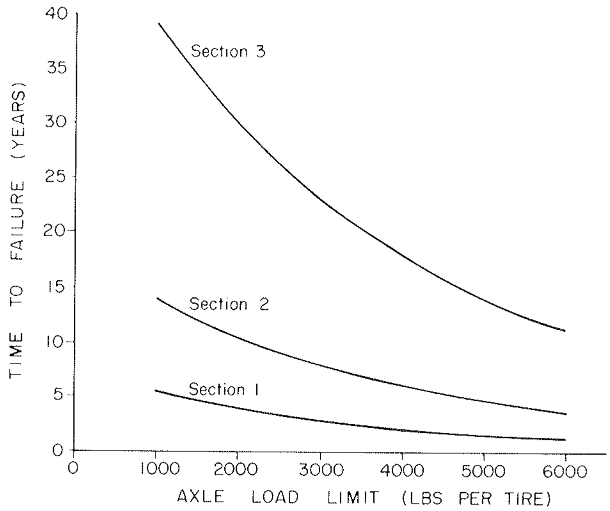


FIGURE 11 Pavement time to failure as a function of tire load limit.

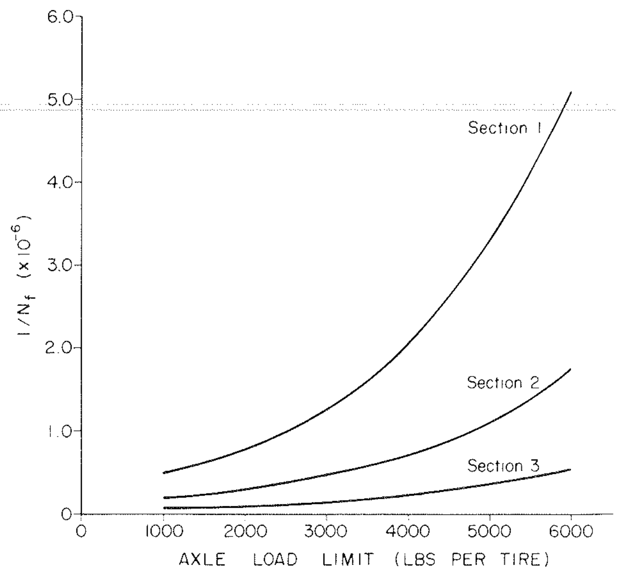


FIGURE 12 Pavement damage responsibility as a function of tire load limit.

indicate that in Section 1 the damage caused by each load application is far more sensitive to the load magnitude than in Section 2 or 3. The marginal damage (rate of increase in pavement damage) is represented by the slope of the curves shown in Figure 12.

Cost-Allocation Considerations

In cost-allocation procedures where costs are assessed to road users according to their associated damage responsibility, the marginal damage is often used as a method to determine marginal cost for a particular axle load. If this methodology were used with the information from Figure 12, the cost allocation determined from marginal damage would be much higher for Section 1 than for Section 3. This would indicate that heavy trucks operating on thin pavements should be assessed higher costs than the same trucks operating on thick pavements.

However, what is not being taken into consideration in the above methodology is the initial construction cost. Section 3 may have small marginal damage associated with heavy load applications, but Section 3 may have been far more costly to construct than Section 1, so the higher marginal cost of maintenance in Section 1 may be offset by the higher initial cost of Section 3. This emphasizes the fact that total cost, and not simply rehabilitation and user costs, should be considered when determining cost responsibility.

SUMMARY AND CONCLUSIONS

The Commonwealth of Pennsylvania has 44,000 mi of roads under its jurisdiction. About two-thirds of these are secondary roads which, in other states, would be the responsibility of local governments. Because the majority of these roads have pavements with limited structural capacity, the state has the authority to restrict axle loads if it is believed that those axle loads would result in excessive damage to the pavement structure. In Pennsylvania, the posting of load limits below the legal maximum occurs on a year-round basis for some roads, as well as on a seasonal (spring load restriction) basis for others.

The main purpose of this research project was to develop rational guidelines for the posting of load limits in Pennsylvania. To evaluate the effect of axle loads under a variety of conditions, a theoretical analysis was conducted that considered various load magnitudes and configurations for different pavement thicknesses and material properties. In this analysis the effect of subgrade strain was studied, using the elastic-layer program BISAR. It was found that axle configuration (i.e., single-, tandem-, and triple-axle assemblies) did not significantly affect pavement response, provided that the load per tire remained the same.

Following the analysis of axle loads, a performance model based on compressive strain at the top of the subgrade was developed. To accommodate Pennsylvania's deflection-measuring equipment, a procedure was developed that estimates the subgrade strain from measurements taken with either the road rater or the FWD. In this way, the new procedure uses deflection measurements to predict pavement performance for a given level of traffic.

A microcomputer program was written that incorporates the new procedure and includes a default traffic stream that is typical of secondary roads. The program generates information concerning predicted years to failure for different load limits. This enables the user to quantitatively consider the effects of axle load limits on pavement deterioration.

In addition to the microcomputer program, a simple figure was developed to allow engineers to estimate the effects of imposing different load limits in the absence of deflection measurements. A second figure was developed that indicates the portion of the pavement damage that caused by a particular axle load. With this information, the engineer can determine the appropriate charges to be assessed to heavy haulers for permits and bonds.

An example application was presented wherein a load-limit analysis was conducted on three different pavement sections. The analysis indicated more damage responsibility for heavy loads on thin pavements than on thick pavements, as would be expected. However, cost allocation based on marginal pavement damage can be misleading if the initial cost of construction is not considered.

The load-limit analysis procedure presented in this paper can be a valuable tool in the evaluation of axle load limits and axle-damage responsibility.

ACKNOWLEDGMENT

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Selected Results from the First Three Years of the Oregon Automatic Monitoring Demonstration Project

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Until the 1980s, the majority of highway traffic data was obtained manually. However, with the evolution of microcomputers, cost-effective automatic data-collection equipment has been implemented. A comprehensive system is made up of weigh-in-motion, automatic vehicle classification, and automatic vehicle identification. Weigh-in-motion determines axle and vehicle weight at full speed on the highway, automatic vehicle classification classifies the traffic into groups (19 in Oregon) by identification of axle spacings, and automatic vehicle identification acts as an "electronic license plate," which can be used with weigh-in-motion and automatic vehicle classification to characterize individual vehicles. These new technologies enable continuous and relatively accurate monitoring of traffic, and therefore lead to improved planning, pavement design, and other activities that use the data. Oregon State Highway Division is a leader in demonstrating automatic vehicle monitoring, which was initiated in the state in 1983. Data are collected in unprecedented amounts at five sites on Interstate 5 (I-5). Oregon State University has developed prototype BASIC software to process the weekly data from the busiest site in tabular or graphical form, designed to enable data to be distributed in the various units in the highway division. Selected results are included in this paper, and other data are presented that show comparison of weights obtained with weigh-in-motion and with static scales. The advantages of having automatic vehicle monitoring data are demonstrated. In particular, the continuous monitoring of the traffic stream completely defines daily, weekly, and seasonal traffic patterns, and clearly indicates growth.

For many years highway vehicle data have been collected for different purposes. Data concerning truck and car volumes are used in transportation planning. Truck gross and axle weight data are needed for weight enforcement and pavement design. Obtaining these data is not a simple task. Vehicle counting was originally done using simple manual counters that required substantial manpower. With the advent of pneumatic tube counters, vehicle counting became much easier and less expensive; however, this method had many limitations for vehicle classification. Truck-weight information has traditionally been obtained from weigh stations where trucks must be stopped and weighed statically. Because these methods of traffic and weight data acquisition were lengthy and costly, statistical data were usually based on short-term sample data. Data obtained in this manner are not reliable because bias is introduced into sample data by the manual data collection methods and the lack of continuously open weigh stations.

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In recent years a new approach has revolutionized vehicle data acquisition technology. In-motion weighing of vehicles at normal highway traffic speeds has become possible at reasonable cost. Induction loops can be used for counting and as part of a classification system (that also needs axle sensors), and automatic identification of vehicles has become a reality. The integration of weight-in-motion (WIM), automatic vehicle classification (AVC), and automatic vehicle identification (AVI) systems in one site provides continuous and accurate data that can be used for a variety of purposes, which include

1. Size, weight, and speed enforcement;
2. Transportation planning;
3. Pavement design and management;
4. Truck fleet management; and
5. Vehicle taxation.

It is significant to note that low-cost WIM and AVC devices have been identified as vital to the Long-Term Pavement Performance (LTPP) element of the Strategic Highway Research Program (SHRP) *(1)*, for which traffic data for hundreds of sites will be required.

In the future an integrated system of many sites in a network will provide a much more powerful means of providing enough information for hazardous material monitoring and crime detection.

Oregon State Highway Division (OSHD) initiated a program in 1983 to evaluate WIM, AVC, and AVI. OSHD currently has five sites in which AVC and AVI are operational and two in which WIM, AVC, and AVI are operational. In addition, OSHD has a portable WIM device that requires installation at suitable bridges. Details of the entire Oregon automatic vehicle monitoring (AVM) program have been described previously by Krukar and Henion *(2)*.

PURPOSE AND SCOPE

The purpose of this paper is to present some results from Oregon's WIM/AVC/AVI demonstration project.

Included in the paper are data applying mainly to Oregon's Jefferson site, but data from the other sites are also included. The Jefferson site on I-5 northbound was chosen for the initial development of data-reduction procedures because it is the only high-speed WIM/AVC/AVI installation and is operated continuously. This site obtains data for both northbound lanes. Data were first collected at this site in April 1984, and presented in this paper will be data collected since that time. The

early development of software for post-processing of the data collected at the Jefferson site has been described by Mohseni *(3)*. Bell and Mohensi *(4)* have described subsequent work. The development of software is an ongoing effort, reflecting the continuing development of AVM technology and its applications.

RESULTS FROM THE JEFFERSON SITE

The WIM system at this site is an International Road Dynamics (IRD) Automatic Highway Scale. The AVC system is made up of two loops connected to a DEC LS1-11/2 computer at the roadside. A microwave AVI system can identify those trucks that have installed an electronic license plate (ELP) voluntarily. At present, about 200 trucks are fitted with ELPs.

The tables output form from the Jefferson system is converted to numeric data files that are then used to produce weekly plots and tables. Numeric files are also used to create cumulative files that contain data for several weeks and are used to produce cumulative plots and tables. Vehicle classifications used in the Oregon weigh-in-motion study are shown in Figure 1. These classifications are based on vehicle axle arrangement and length.

Data Collection

The following primary (raw) data are collected by the WIM/AVC/AVI system on the passage of each vehicle:

1. Time and day of pass by roadside unit,
2. Vehicle license plate by AVI system,
3. Vehicle length by AVC system,
4. Axle spacing and number of axles by AVC system, and
5. Weight of individual axle (by WIM).

These data are then processed by the roadside computer to produce the secondary "cooked" data listed as follows:

1. Vehicle axle arrangement,
2. Vehicle classification based on axle arrangements,
3. Axle and gross vehicle weight, and
4. AASHTO rigid and flexible Equivalent Single Axle Load (ESAL).

The roadside computer outputs the data in two forms: tables and view. The view data consist of primary and secondary data for each vehicle and can be accessed at the time of passage. These data are not stored in the roadside computer because of

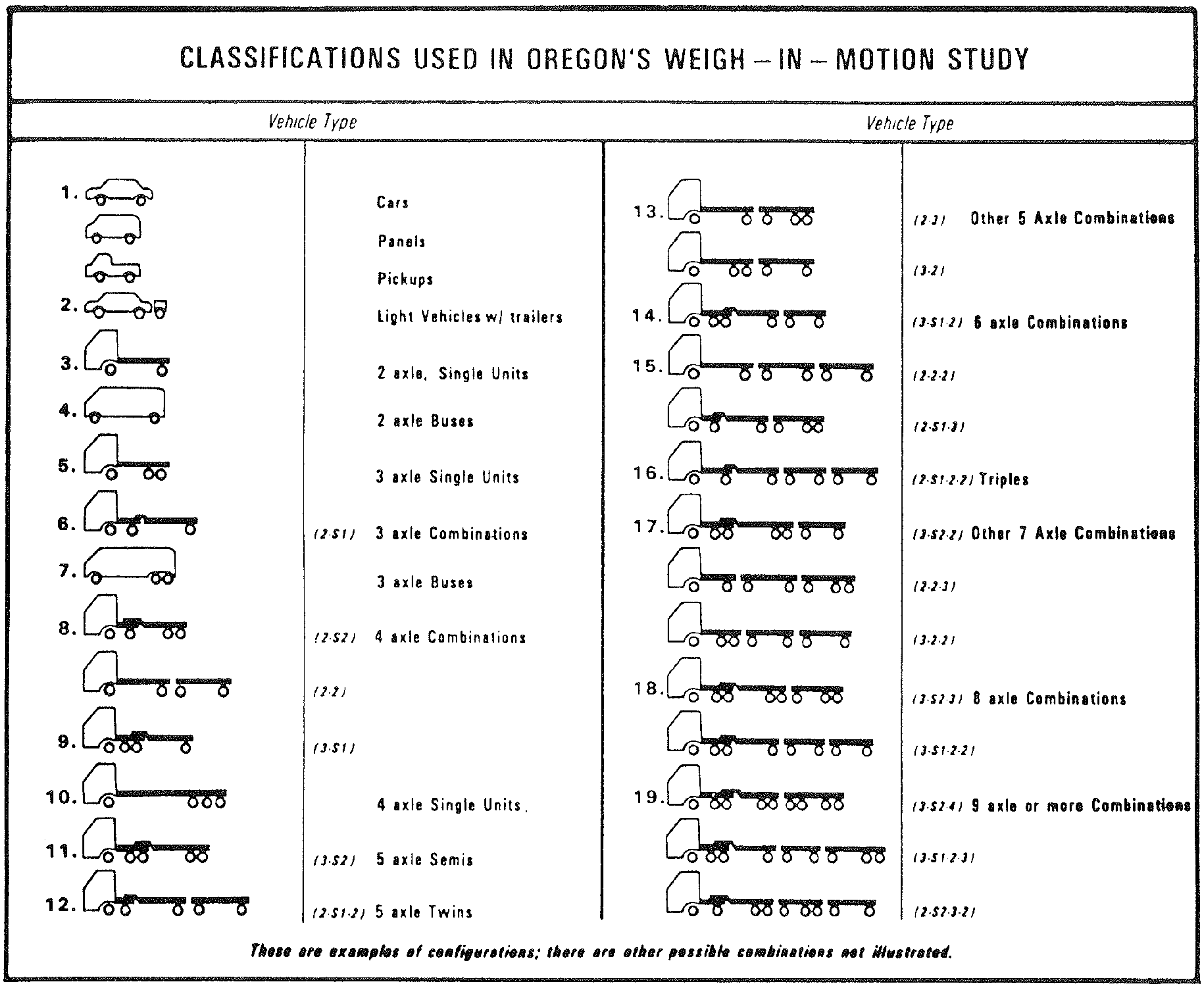

FIGURE 1 Classifications used in Oregon's weigh-in-motion study.

the large memory size needed. Rather, the raw and cooked data are processed into 11 tables in the form of a report, referred to as the tables output form. The following are the titles of the tables:

1. Most Recent Vehicles With Transponders (ELPs),
2. Weight Distribution and Average 18-k ESAL by Vehicle Type,
3. Numbers of Truck Axles by Weight,
4. Vehicles with the Highest Flexible 18-k ESAL,
5. Average Vehicle Length in Feet by Type,
6. Number of Vehicles and 18-k ESAL by Day of the Week (Lane 1),
7. Number of Vehicles and 18-k ESAL by Day of the Week (Lane 2),
8. Cars and Single Unit Truck Volume by Hour and Day of the Week,
9. Five-Axle Semis and Other Truck Volume by Hour and Day of the Week,
10. Traffic Volume by Speed Range, and
11. Five-Axle Semis (Type 11) Flexible 18-k ESAL.

Note that the first five tables are cumulative tables and provide data for a desired period of time (usually 1 week), which is controlled by OSHD from a remote computer. The rest of the tables are daily tables and contain data for each day of the week beginning on Mondays at 00.00 hr. Tables 1 and 2 are examples of the third and fourth tables listed in the table titles. Both of these provide detailed axle load data.

Data Communication and Storage

The tabular weekly reports (tables) that are collected at the Jefferson site are transferred to the OSHD Economic Services unit in Salem via modem. Other users can also access the data. To transfer the reports, the computer operator in Salem calls the on-site computer every Monday morning and downloads the reports onto an IBM-AT hard disk. Any communications software can be used to download the reports.

Reports have been obtained by Oregon State University from ODOT since April 7, 1984, and stored on an IBM-XT hard disk. In order to have continuous data, the report from an adjacent week was used whenever the report for a week was not available or was incomplete (about 10 percent of all weeks). Presented in this paper are data obtained through September 1986 (130 weeks).

Procedure for Reducing Data

The view and tabular output are originally in report form (i.e., output file form). Thus, the first task is to convert the tables into

TABLE 1 NUMBERS OF TRUCK AXLES BY WEIGHT

Front Axles Weight (kips)	#	Single Axles Weight (kips)	#	Tandem Axles Weight (kips)	#	Tridem Axles Weight (kips)	#
< 4	1947	< 4	1766	< 8	707	< 8	9
4-5	628	4-5	1179	8-10	713	8-10	2
5-6	303	5-6	1164	10-12	918	10-12	10
6-7	354	6-7	1053	12-14	1093	12-14	4
7-8	691	7-8	967	14-16	756	14-16	2
8-9	2223	8-9	846	16-18	700	16-18	3
9-10	3240	9-10	754	18-20	702	18-20	2
10-11	3105	10-11	624	20-22	651	20-22	1
11-12	1960	11-12	585	22-24	686	22-24	3
12-13	513	12-13	637	24-26	669	24-26	1
13-14	55	13-14	594	26-28	670	26-28	2
14-15	9	14-15	563	28-30	811	28-30	2
15-16	5	15-16	638	30-32	1383	30-32	2
16-17	3	16-17	667	32-34	2292	32-34	1
17-18	2	17-18	723	34-36	2896	34-36	1
18-19	0	18-19	752	36-38	1706	36-38	4
19-20	0	19-20	618	38-40	451	38-40	1
20-21	0	20-21	482	40-42	71	40-42	2
21-22	1	21-22	325	42-44	10	42-44	0
22-23	0	22-23	141	44-46	7	44-46	1
23-24	0	23-24	65	46-48	2	46-48	1
24+up	0	24+up	25	48-50	3	48-50	1
				50+up	3	50-52	0
						52-54	1
						54-56	0
						56-58	0
						58-60	0
						60-62	0
						62-64	0
						64-66	0
						66-68	0
						68-70	0
						70-72	0
						72-74	0
						74+up	0
Overweight Axles:	n/a		1038		5149		n/a
Total Axles:	15039		15168		17900		56
Average Weight:	8.7		10.8		25.9		20.8
Percent Overloads[a]	n/a		6.0		28.3		n/a

Note: Oregon State Highway Division, Interstate 5, Jefferson Site, from Monday, June 24, 1985, at 8:10 a.m. to Monday, July 1, 1985, at 8:06 a.m.

[a]These estimates are based on WIM data, not static weights.

TABLE 2 VEHICLES WITH HIGHEST FLEXIBLE 18-K ESAL

					Axle (or Axle Group) Weights						Axle	Gross		18-K ESAL	
#	Type	Lane	Day	Time	1st	2nd	3rd	4th	5th	6th	Configuration	Weight	Speed	Rigid	Flexible
1	12	1	Tue Jun 25	13:01	8.7	25.4	22.7	23.1	21.6		11111	101.5	56	12.11	10.99
2	16	1	Sun Jun 30	02:24	9.4	22.4	20.5	21.3	16.3	22.5	1111111	129.1	56	10.44	9.87
3	16	1	Mon Jun 24	15:42	8.1	22.2	23.5	20.4	21.1	15.5	1111111	126.5	48	10.39	9.77
4	12	1	Thu Jun 27	10:59	9.1	22.1	24.6	18.7	23.8		11111	98.3	57	10.66	9.76
5	12	1	Mon Jun 24	15:29	10.0	21.2	24.3	19.5	24.1		11111	99.1	54	10.52	9.66
6	11	1	Thu Jun 27	20:48	8.9	47.2	52.0				122	108.1	55	17.43	9.39
7	12	1	Tue Jun 25	15:23	9.7	21.9	21.3	20.7	24.8		11111	98.4	60	10.17	9.37
8	12	1	Mon Jun 24	08:55	9.3	22.6	21.1	23.1	22.3		11111	98.4	56	10.10	9.34
9	16	1	Sat Jun 29	02:56	9.9	22.1	23.5	18.0	18.8	15.2	1111111	127.0	57	9.73	9.23
10	15	1	Mon Jun 24	08:10	11.5	42.6	54.7	4.6			1221	113.4	62	16.74	9.07
11	12	1	Wed Jun 26	15:17	9.5	20.5	21.6	19.1	25.6		11111	96.3	59	9.74	8.96
12	12	1	Thu Jun 27	19:50	8.8	19.6	23.2	20.2	24.0		11111	95.8	54	9.46	8.76
13	12	1	Wed Jun 26	19:41	8.6	22.3	23.7	18.0	22.3		11111	94.9	59	9.24	8.57
14	16	1	Wed Jun 26	15:02	8.3	19.6	18.1	19.7	21.9	20.6	1111111	124.9	48	8.85	8.52
15	12	1	Fri Jun 28	13:04	9.7	20.8	23.0	18.7	23.6		11111	95.8	58	9.16	8.50
16	11	1	Wed Jun 26	21:47	8.5	45.9	50.2				122	104.6	36	15.57	8.42
17	19	1	Tue Jun 25	14:44	11.3	20.8	43.1	44.8	40.0		12222	160.0	55	15.10	8.38
18	12	1	Sat Jun 29	17:38	9.4	22.5	21.6	21.4	20.8		11111	95.7	58	8.83	8.27
19	14	1	Wed Jun 26	20:28	8.9	26.8	24.9	19.1	23.9		12111	103.6	63	9.34	8.25
20	12	1	Fri Jun 28	11:09	9.2	21.2	23.3	17.9	22.8		11111	94.4	59	8.82	8.21
21	12	1	Thu Jun 27	08:13	10.5	23.6	22.7	19.8	19.0		11111	95.6	58	8.76	8.17
22	13	1	Mon Jun 24	10:49	11.1	45.4	22.5	22.1			1211	101.1	55	11.34	8.15
23	14	1	Mon Jun 24	12:10	9.2	30.3	20.4	23.8	23.4		12111	107.1	62	9.29	8.12
24	12	1	Mon Jun 24	15:29	8.7	21.8	23.1	18.7	21.8		11111	94.1	55	8.67	8.10
25	12	1	Mon Jul 1	06:27	9.2	25.3	19.3	18.5	20.4		11111	92.7	63	8.51	7.89
26	12	1	Mon Jun 24	16:57	9.4	20.5	23.1	17.9	22.6		11111	93.5	58	8.35	7.81
27	12	1	Tue Jun 25	15:23	9.3	21.9	22.4	18.5	21.4		11111	93.5	60	8.16	7.67
28	12	1	Wed Jun 26	15:17	9.5	19.9	22.8	18.1	22.9		11111	93.2	57	8.18	7.67
29	12	1	Mon Jun 24	22:40	8.8	23.0	18.6	19.3	22.8		11111	92.5	57	8.19	7.66
30	17	1	Fri Jun 28	16:19	11.5	32.9	23.2	23.0	15.1	16.4	121111	122.1	57	8.80	7.62
31	17	2	Tue Jun 25	15:51	12.3	32.5	19.0	22.1	19.7	20.2	121111	125.8	59	8.61	7.62
32	12	1	Fri Jun 28	15:46	8.9	21.2	21.8	21.2	20.3		11111	93.4	58	8.07	7.60
33	15	1	Thu Jun 27	03:26	8.3	26.8	38.0	34.6			1122	107.7	55	10.47	7.53
34	16	1	Sun Jun 30	02:14	9.4	20.6	19.6	17.4	17.1	20.6	1111111	122.2	56	7.72	7.52
35	16	1	Wed Jun 26	22:47	9.0	23.4	21.1	19.1	19.6	9.0	1111111	108.9	58	7.95	7.49
36	19	2	Mon Jun 24	14:54	9.8	17.6	38.8	42.1	43.7		12222	152.0	52	13.45	7.47
37	14	1	Wed Jun 26	11:51	8.4	28.7	20.7	22.0	23.8		12111	103.6	57	8.43	7.47
38	12	1	Fri Jun 28	14:14	9.0	23.9	21.8	18.3	18.8		11111	91.8	55	7.94	7.44
39	12	1	Thu Jun 27	20:50	8.8	21.1	22.5	17.3	22.0		11111	91.7	57	7.84	7.37
40	12	1	Thu Jun 27	14:20	9.9	22.4	19.2	19.8	21.9		11111	93.2	58	7.78	7.35

Note: Oregon State Highway Division, Interstate 5, Jefferson Site from Monday, June 24, 1985, at 8:10 a.m. to Monday, July 1, 1985, at 8:06 a.m. ESAL = equivalent standard axle load.

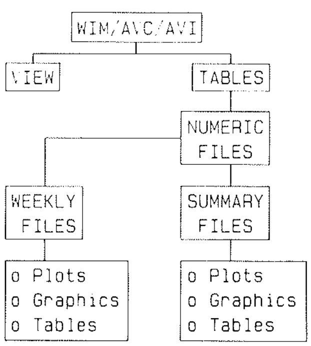

FIGURE 2 Data processing and presentation.

numeric form so that data (numbers) can be read individually as a data file. This is done by the CHEWALL BASIC program, which enables the user to create the numeric files for a number of weeks. For each table in each week one numeric file is created. WIM Tables 1 and 4 are not converted to numeric files because the data in these tables are only useful in the report form. Thus, nine numeric files are created for each week of the year. These numeric files are the source of data for weekly and cumulative summary tables and plots. Shown in Figure 2 is a flowchart depicting the data-reduction process.

Weekly Summary Tables and Plots

The data in the 11 tables are reduced and summarized in 3 summary tables and in 19 plots for each week. This is done by the WKMENU computer program written in IBM BASIC. Three summary tables summarize the weekly data for different applications. The user can select the desired plot for plotting and summary tables for printing. Selected plots for 1 week are shown in Figures 3 through 10. Table 3 shows summary table information for vehicle volumes and ESALs.

Cumulative Plots and Tables

The data in the numeric files are used to produce cumulative summary files. This is done by using the TABMENU computer program, which reads data from the previously created numeric files for a specified number of weeks and prints the data into a cumulative summary file. Thus, a summary file contains data for several weeks. There are 21 choices of summary files, as follows:

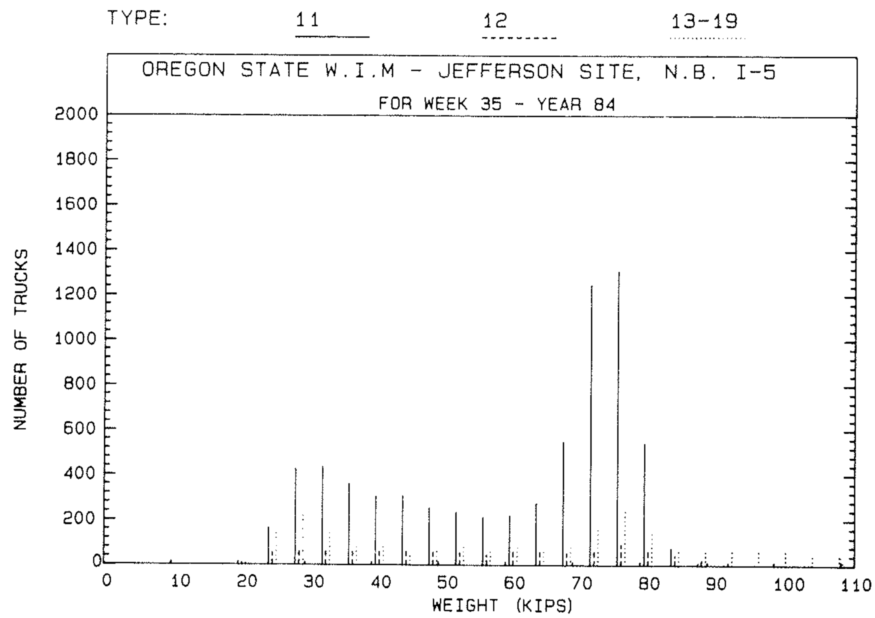

FIGURE 3 Weight distribution by vehicle type.

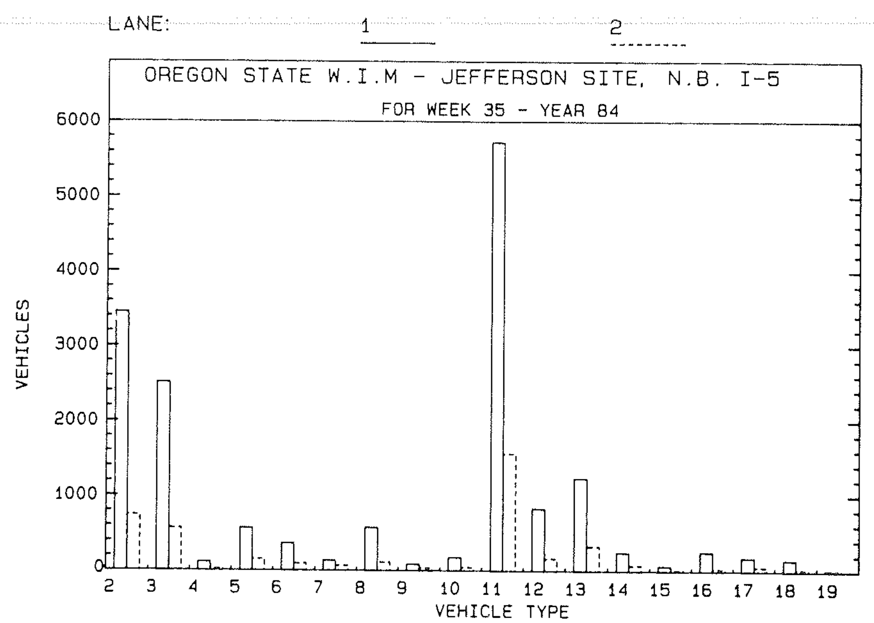

FIGURE 4 Average weekly volume by type and lane.

1. Average Weight of Light Trucks (Both Lanes),
2. Average Weight of Heavy Trucks (Both Lanes),
3. Average ESAL for Light Trucks (Both Lanes),
4. Average ESAL for Heavy Trucks (Both Lanes),
5. Weekly Axle Volume by Type (Both Lanes),
6. Average Axle Weight by Type (Both Lanes),
7. 5 Axle Semis Front Axle Weight—Lane 1,
8. 5 Axle Semis Front Axle Weight—Lane 2,
9. Truck and Vehicle Weekly Volume (Both Lanes),
10. Weekly Truck Volume by Type (Both Lanes—Types 3–19),
11. Weekly Truck Volume by Type (Both Lanes—Types 12–19),
12. Percent Trucks in Vehicles by Lane,
13. Percent Vehicles not Weighed by Lane,
14. Percent Truck Types in Trucks (Both Lanes),
15. Percent Vehicles in Lane 2,
16. Weekly ESAL by Lane (Both Lanes),
17. Weekly ESAL by Truck Class (Both Lanes—Types 3–19),
18. Weekly ESAL by Truck Class (Both Lanes—Types 12–19),
19. Percent Weekly ESAL by Truck Class (Both Lanes),
20. Percent ESAL in Lane 2 by Type, and
21. Average Weekday Speed (Both Lanes).

Note that all ESALs are for flexible pavement and those parts of the title in parentheses are omitted on the plots and tables.

Summary files should be checked for errors and aberrations because inconsistencies in WIM operation and modem

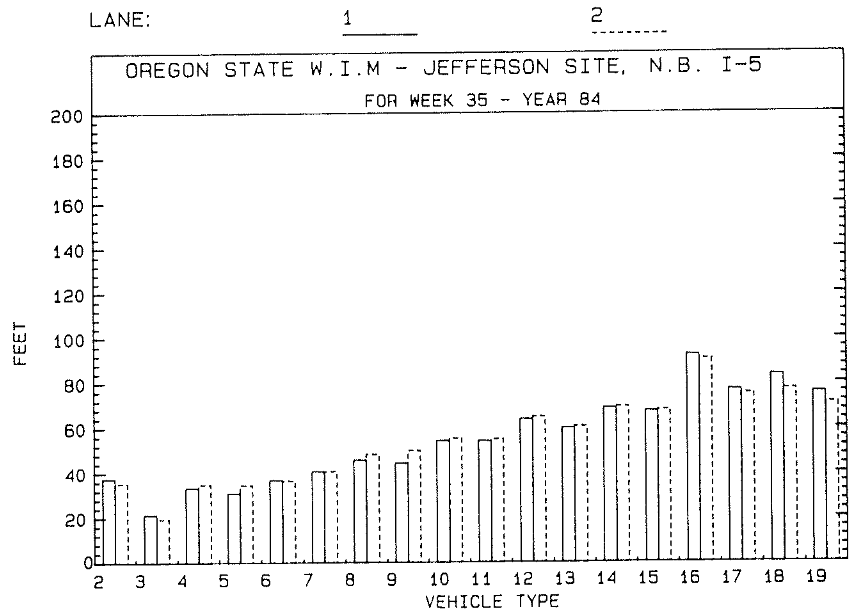

FIGURE 5 Average weekly length by type and lane.

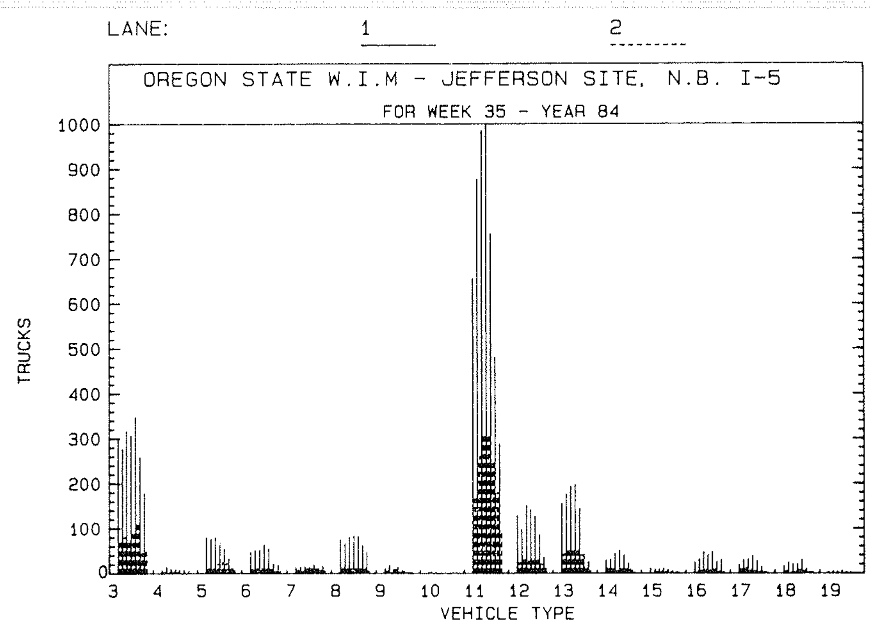

FIGURE 6 Daily truck volume by type.

communication may mean that values may not be representative of the data for some of the weeks. This can be done by an editor's program such as SPFPC, or by suitable word processing software.

Once summary files are corrected, they can be printed by TABMENU or plotted by using the PLOTMENU computer program. Menu options are used for both programs. To plot the summary files, an HP 7475A series plotter is used. An option is to show the plot graphically on the computer screen and then to copy the graphic to a printer. Figures 11 through 22 show selected summary plots. Figure 12 is plotted from an extended version of the example summary file (for average ESALs of heavy trucks), shown in Table 4. Illustrated in this table is an example of the result of incomplete data; week 51 data were incomplete, and were replaced with week 52 data, which show low ESALs because of the holiday season.

Limitations of the Tables and Plots

Owing to deliberate or accidental misses, about 20 percent of vehicles are not weighed by the system, with an average of about 14 percent within that 20 percent being unclassified. Deliberate misses are attributed to about 5 percent of vehicles, and the remaining misses are due to lane changes at the site. The majority of the data in the weekly reports represents either classified (about 86 percent) or weighed (about 80 percent) vehicles. In fact, only two of the tables in the weekly reports

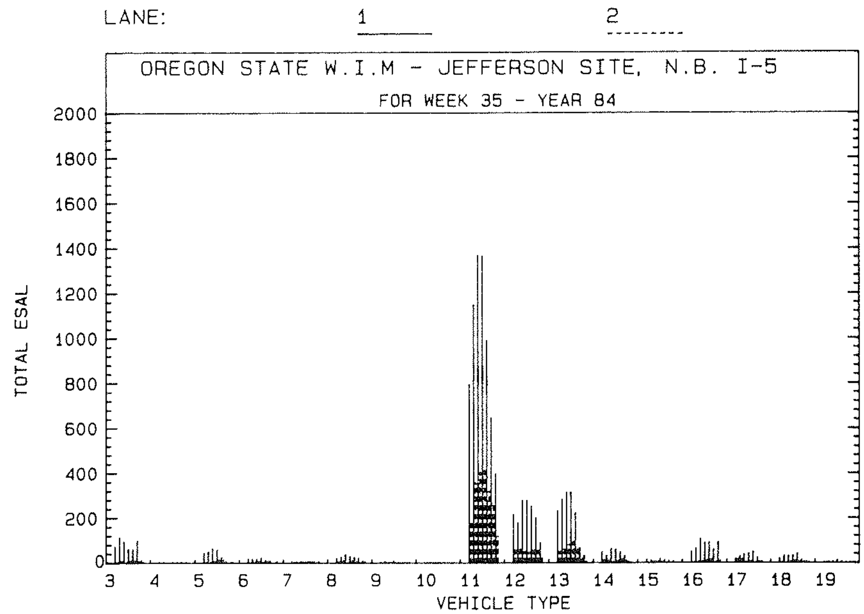

FIGURE 7 Daily total truck ESAL by type.

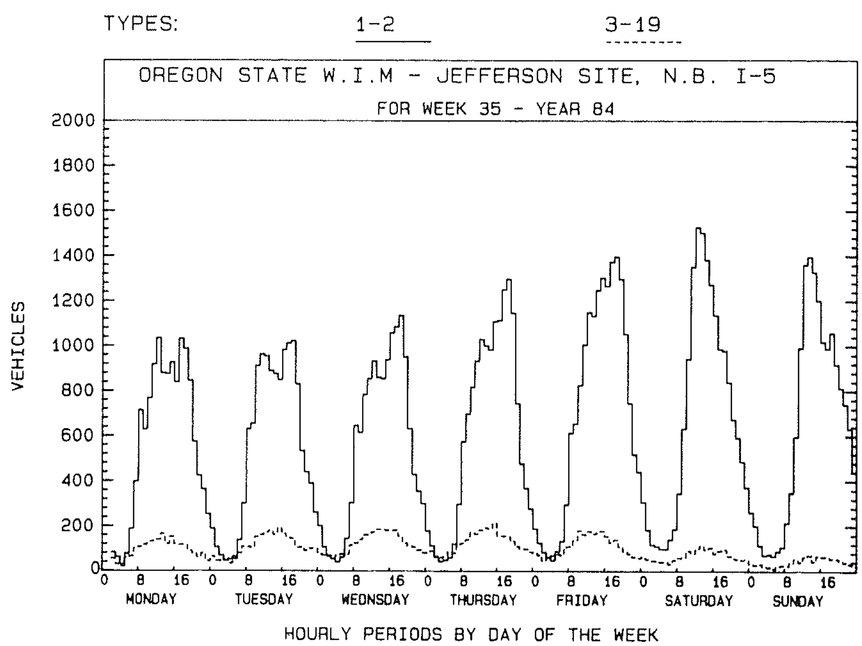

FIGURE 8 Hourly vehicle volume by class and day of week.

indicate the total vehicles not weighed each day, and clearly there can be no data indicating the classification of those vehicles not weighed or not classified.

For this reason, all of the plots that can be developed represent only a portion of the total traffic. No attempt has been made as yet to adjust the data, as accurate adjustment factors cannot be developed except when considering the total traffic. However, the weekly summary tables (e.g., Table 3) do present adjusted data, assuming that all unclassified and unweighed vehicles are evenly distributed among the 19 vehicle classifications. The cumulative summary tables contain no such adjustment at the present time.

COMPARISON OF TRUCK WEIGHTS OBTAINED AT DIFFERENT SITES

The Jefferson WIM site is located approximately 30 mi south of the Woodburn weigh station, also on I-5, where a WIM sorter system is in use. The sorter system is used to expedite passage of legally loaded vehicles through the station, but causes those vehicles close to or in excess of the statutory limits to be directed to static scales for traditional weighing. There are approximately 200 Oregon trucks voluntarily fitted with electronic tags for automatic identification at Jefferson,

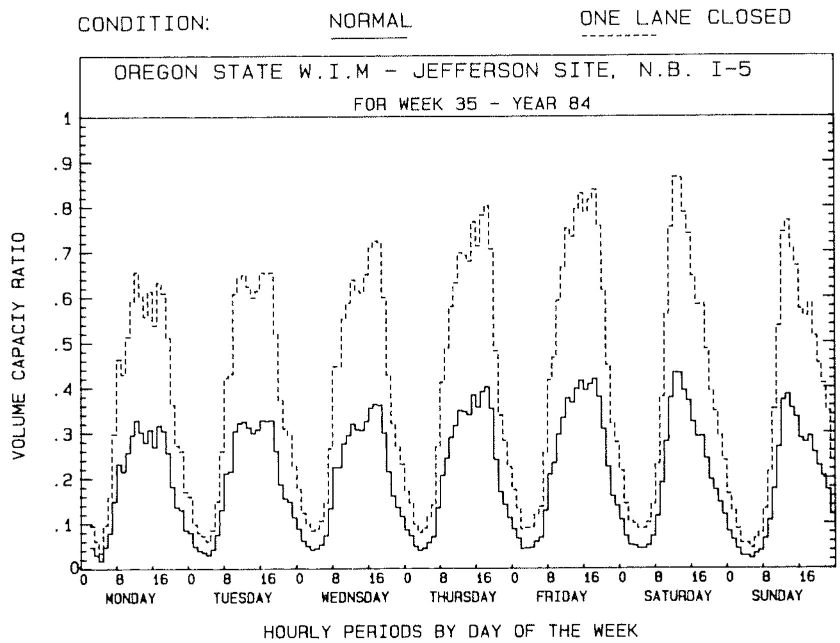

FIGURE 9 Hourly volume capacity ratio.

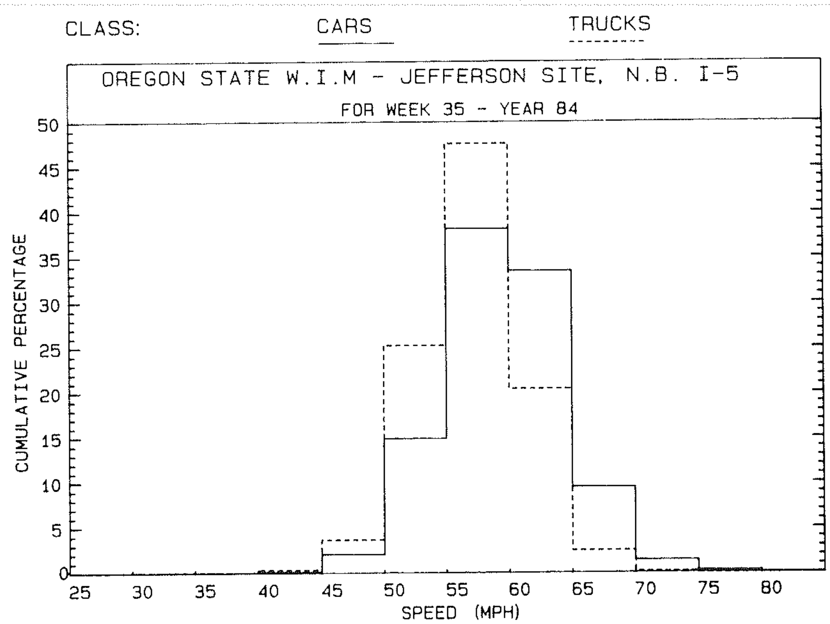

FIGURE 10 Average weekday speed distribution.

Woodburn, and four other locations on I-5. This allows detailed comparison of their axle and gross loads, as they will be weighed at both WIM sites and on the static scales (if requested as part of a short study) provided they travel through the I-5 corridor.

Table 5 shows data from a recent study in which Jefferson WIM data were compared with Woodburn static data for Type 11 trucks. This study considered all trucks rather than just those with tags, and was accomplished by matching Public Utility Commission (PUC) plate numbers at each location. The data show that the mean gross loads measured at Jefferson in Lane 1 are 5 percent higher than the static loads. This difference is used elsewhere in this paper in estimating overloading. However, it should be noted that the differences vary with axle type, gross weight, and lane.

DISCUSSION OF DATA COLLECTED

Some significant aspects of the data are highlighted in the following paragraphs.

Weekly Data

Vehicle Length

It can clearly be seen in Figure 5 that Type 16 trucks are the largest trucks using I-5, averaging about 90 ft long. This is as expected, because Type 16 is a 2-S1-2-2 triple-trailer vehicle (see Figure 1). The 2-S2-2-2 triple trailer included in Type 18 could be longer, but is less frequent than the 3-S2-3 truck, and therefore is not reflected in the average length of Type 18.

TABLE 3 ADJUSTED WEEKLY TRUCK DISTRIBUTION BY TYPE

Type	Description	No. of Vehicles	Percent Vehicles	Average ESAL	Total ESAL
1	Cars	109477	82.2	0.00	0.00
2	Cars+Trailers	4792	3.6	0.00	0.00
3&4	Rigid 2-Axle	3590	2.7	0.12	430.75
5&7	Rigid 3-Axle	816	0.6	0.42	342.74
10	Rigid 4-Axle	0	0.0	0.00	0.00
6	3-Axle Semi	515	0.4	0.31	159.53
8&9	4-Axle Semi	1059	0.8	0.33	349.63
11	5-Axle Semi	8560	6.4	1.62	13867.69
12	5-Axle Twin	1188	0.9	2.45	2910.91
13-19	Other	3244	2.4	1.80	5839.22
3-19	Total (Trucks)	18972	14.2	1.26	23900.45
1-19	Total (All)	133241	100.00	0.18	23900.45

Note: From Monday, June 24, 1985, at 8:10 a.m. to Monday, July 1, 1985, at 8:06 a.m. ESAL = equivalent standard axle load.

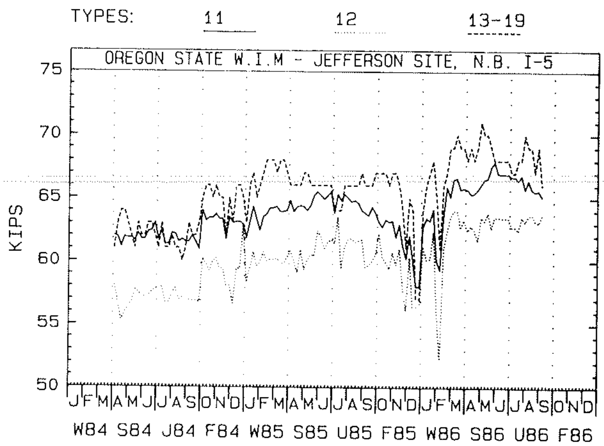

FIGURE 11 Average weight of heavy trucks.

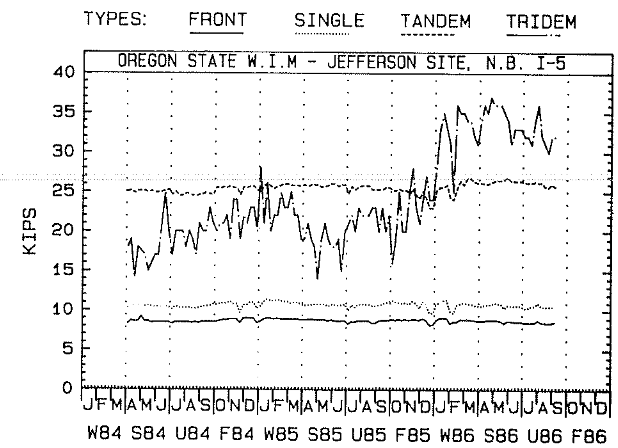

FIGURE 13 Average axle weight by type.

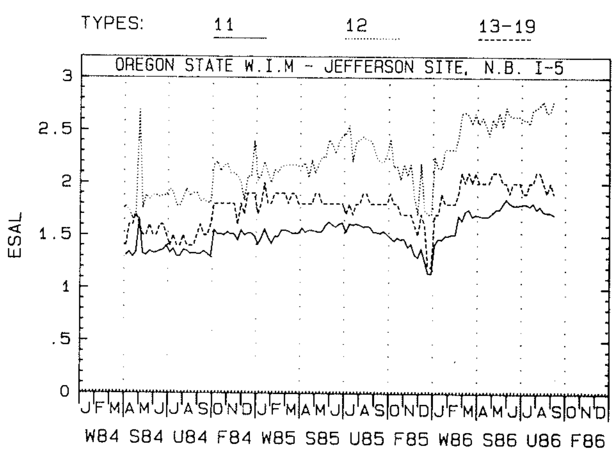

FIGURE 12 Average ESAL for heavy trucks.

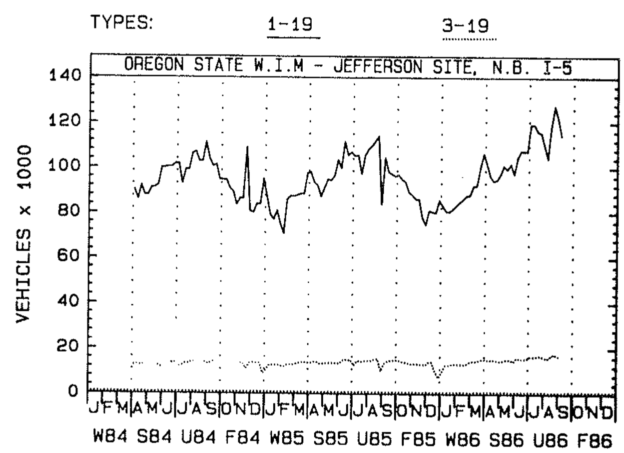

FIGURE 14 Truck and vehicle weekly volume.

Volume of Vehicles by Lane Each Day

It is shown in Figure 4 that, excluding cars, the Type 11 truck (3-S2) is the most frequently occurring vehicle. About half of the truck traffic is this type of vehicle. It can also be noted (see Figure 6) that the peak day for truck traffic is either Wednesday or Thursday.

Hourly Volumes of Cars and Trucks

It can be seen in Figure 8 that, for the week shown, there is no pronounced morning peak hour, but that the afternoon peak for Types 1 and 2 (cars and other light vehicles) is 5:00 to 6:00 p.m., Monday to Friday. The weekend peaks are at about midday. It is also shown in Figure 8 that for the week covered, which was followed by the Labor Day holiday, the heaviest flow occurs on a Saturday. During spring and summer the

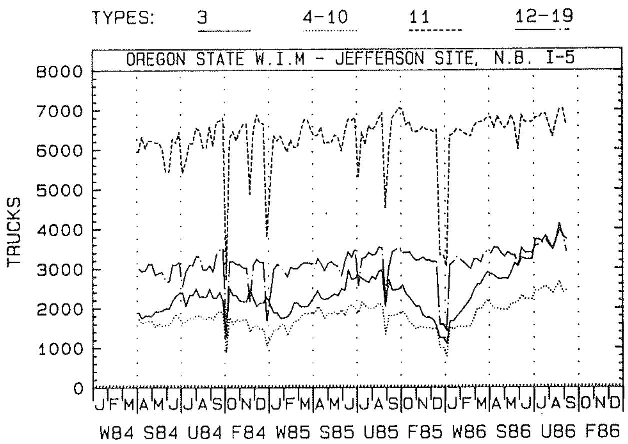

FIGURE 15 Weekly truck volume by type (Types 3; 4–10; 11; 12–19).

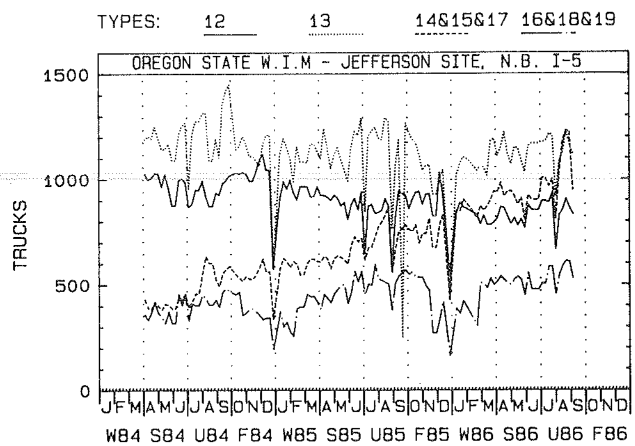

FIGURE 16 Weekly truck volume by type (Types 12; 13; 14, 15, and 17; 16, 18, and 19).

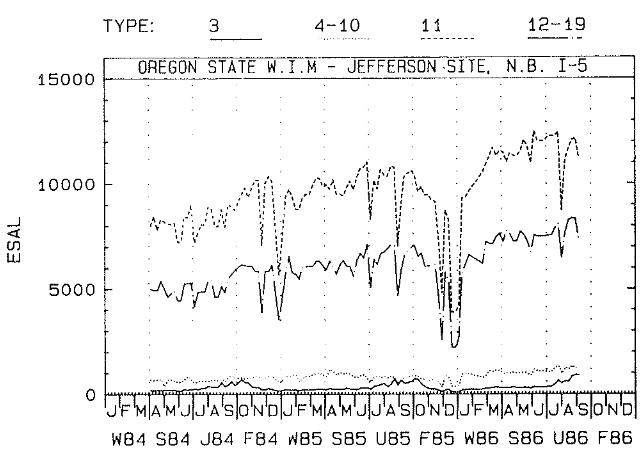

FIGURE 17 Weekly ESAL by truck class (Types 3; 4–10; 11; 12–19).

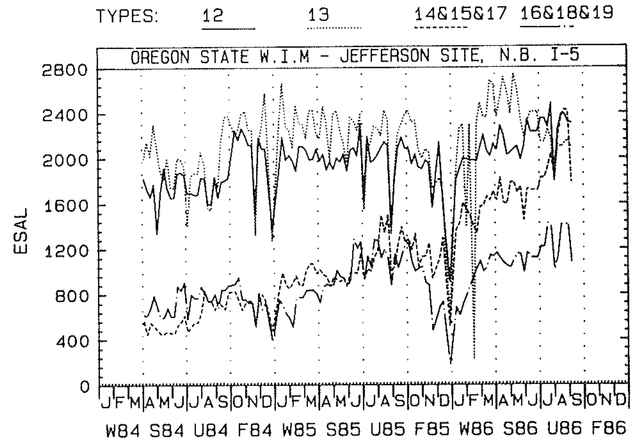

FIGURE 18 Weekly ESAL by truck class (Types 12; 13; 14, 15, and 17; 16, 18, and 19).

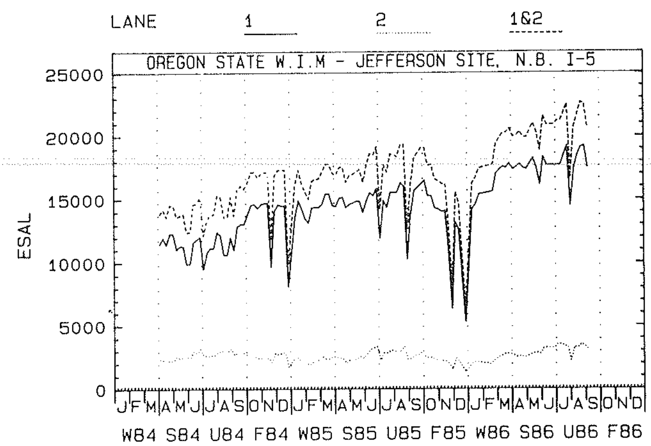

FIGURE 19 Total ESAL by lane.

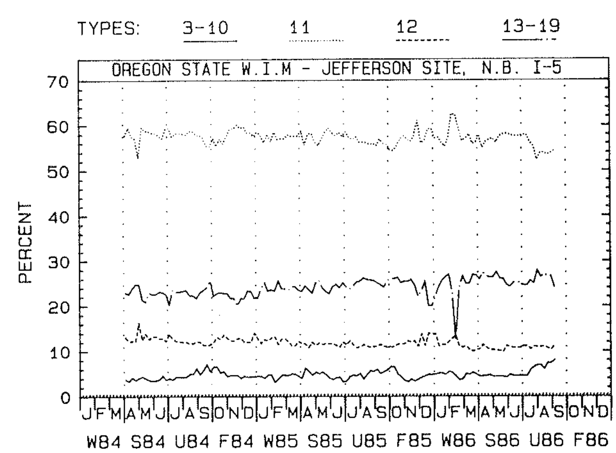

FIGURE 20 Percent weekly ESAL by truck class.

heaviest flows for Types 1 and 2 are consistently observed on Friday, Saturday, and Sunday. During winter and fall, Friday tends to be the busiest day.

Volume Capacity Ratio

Shown in Figure 9 is the demand on the freeway at Jefferson. This is based on the assumption that a single lane has a capacity of 2,000 vehicles/hr under normal operating conditions. With the flow conditions prevailing at Jefferson, there are no 1-hr periods when 0.45 capacity is exceeded for the example shown, assuming one lane closed results in a peak of 0.88 capacity during the Saturday peak hour. To date, there have been no occasions when these values have exceeded 0.50 or 1.00, respectively.

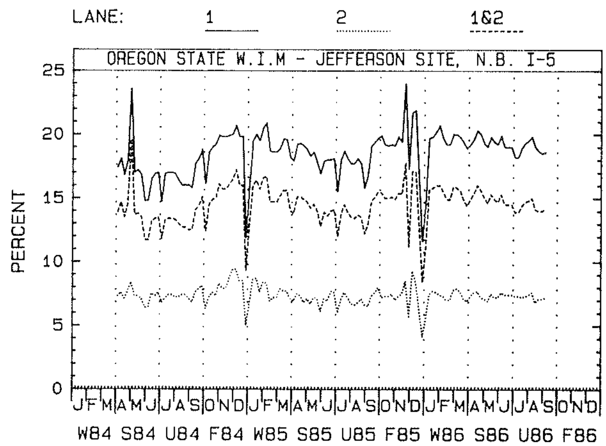

FIGURE 21 Percent trucks in vehicles by lane.

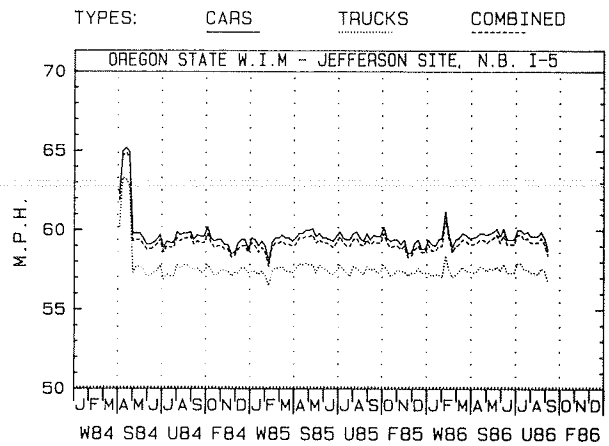

FIGURE 22 Average weekday speed.

TABLE 4 EXAMPLE SUMMARY FILE: AVERAGE ESAL FOR HEAVY TRUCKS

Week	Year,	Types: 11	12	13-19
1	85	1.49	2.40	1.90
2	85	1.41	2.02	1.70
3	85	1.48	2.08	1.80
4	85	1.56	2.20	2.00
5	85	1.48	2.09	1.80
6	85	1.42	2.00	1.80
7	85	1.49	2.12	1.90
8	85	1.49	2.10	1.90
9	85	1.55	2.16	1.90
10	85	1.55	2.16	1.90
11	85	1.54	2.17	1.80
12	85	1.52	2.17	1.90
13	85	1.52	2.17	1.90
14	85	1.52	2.16	1.80
15	85	1.57	2.16	1.80
16	85	1.53	2.19	1.80
17	85	1.55	2.05	1.80
18	85	1.55	2.22	1.80
19	85	1.54	2.09	1.90
20	85	1.53	2.18	1.90
21	85	1.60	2.24	1.80
22	85	1.63	2.41	1.80
23	85	1.60	2.36	1.80
24	85	1.58	2.28	1.80
25	85	1.61	2.39	1.80
26	85	1.62	2.45	1.80
27	85	1.53	2.46	1.70
28	85	1.61	2.55	1.80
29	85	1.60	2.20	1.70
30	85	1.61	2.39	1.80
31	85	1.60	2.41	1.80
32	85	1.58	2.44	1.90
33	85	1.59	2.41	1.90
34	85	1.58	2.41	1.80
35	85	1.53	2.35	1.80
36	85	1.53	2.24	1.80
37	85	1.52	2.20	1.80
38	85	1.54	2.20	1.80
39	85	1.51	2.27	1.80
40	85	1.48	2.42	1.90
41	85	1.45	2.15	1.80
42	85	1.48	2.16	1.80
43	85	1.46	2.03	1.70
44	85	1.47	2.18	1.70
45	85	1.39	2.07	1.70
46	85	1.42	2.19	1.70
47	85	1.32	1.87	1.60
48	85	1.29	1.70	1.50
49	85	1.38	2.19	1.70
50	85	1.26	1.74	1.60
51	85	1.14	1.70	1.20
52	85	1.14	1.70	1.20

Note: ESAL = equivalent standard axle load.

Speed

Speed trends are shown in Figure 10. About 80 percent of the traffic is traveling faster than 55 mph and 20 percent faster than 60 mph. Trucks and cars are traveling at about the same speed, but truck speeds are more uniform. Because most of the traffic is traveling at about the same speed, the operating characteristic at this site is very safe.

Truck Weights and ESALs

It is clearly shown in Figure 7 that the Type 11 truck provides by far the most significant contribution to weekly ESAL for the week shown. Other trucks with 5 or more axles, particularly Types 12 and 13, provide a significant contribution. More recent data show a decline in the percentage contribution of Types 11, 12, and 13, and an increase in that due to Types 14 through 19. This will be discussed further in the section on summary data.

Estimation of Overloading

Table 1 shows that, for the week shown, about 6 percent of measured single axle loads and 28 percent of measured tandem axle loads are above the federal limits of 20,000 lb and 34,000 lb, respectively. As shown in Table 5, for Type 11 trucks the WIM weights for Lane 1 are about 5 percent higher than static weights. Using this difference as a conservative factor to apply to each lane and all axles, estimates of overloaded axles should be reduced by about 50 percent, resulting in about 3 percent and 15 percent respectively for single and tandem axles. Clearly, great care should be taken in calibrating WIM scales and in interpreting data if accurate estimates of overloading are required.

Table 2 gives a clear indication of the types of vehicle providing the heaviest loads and most pavement damage (highest ESALs). This table shows the 40 heaviest vehicles each week (loads should be reduced by about 5 percent to reflect static weights). With the exception of two vehicles, all the trucks are double- or triple-trailer types with predominantly single axles, and with a few exceptions all loads are within 10 percent of the statutory axle load limits. It should be noted that many of the vehicles shown will be operating under permit. The table shows that ESAL values range from about 7 to almost 11 per truck [flexible pavement, structural number (SN) = 5], and the total ESAL from these trucks alone is about 350, or 2 percent of the total ESAL for the week shown.

Adjustments to Data

Shown in Table 3 are adjusted vehicle volumes and ESALs for 1 week. The adjustment is achieved by using data from the weekly tables, which enables the number of vehicles that are

TABLE 5 PERCENT COMPARISON OF DYNAMIC WEIGHTS VERSUS STATIC WEIGHT

				Weight Comparisons - %		
Lane	Sample Size	Heavy Vehicles		Arithmetic Mean	Mean Deviation	Standard Deviation
1	1671	All Trucks	GVW	+5.0	3.7	7.2
			Front Axle	+0.6	4.3	6.2
			Drive Axles	+6.4	5.1	10.3
			Trailer Axles	+5.6	5.4	9.5
	93	Trucks GVW >	70,000 lbs	+5.6	2.5	3.6
			Front Axle	0.0	4.4	6.1
2	422	All Trucks	GVW	-1.6	1.7	4.1
			Front Axle	-6.0	5.2	7.6
			Drive Axles	-1.6	3.8	4.5
			Trailer Axles	-0.4	5.4	7.4
	23	Trucks GVW >	70,000 lbs	-0.5	3.4	4.7
			Front Axle	-7.1	5.9	8.9

Note: Jefferson high-speed weigh-in-motion (WIM) versus Woodburn Static Scale 3S-2 heavy vehicles (five-axle semis, Type 11), October 7, 8, 9, 11, and 15, 1985. 1 = range of heavy vehicle gross vehicle weights (GVWs) at Jefferson WIM from sample size 29,300 lb low to 88,200 lb high. 2 = range of heavy vehicle GVWs at Jefferson WIM from sample size 27,600 lb low to 89,700 lb high.

either not classified or not weighed to be identified. This number is then distributed proportionally among all vehicles. To date this is the only attempt at adjustment that has been made. However, such adjustments will be applied to all plotted data in the future because of the significant differences that result.

Summary Data

Data Variations

All the plots show that there are few consecutive weeks when the observed data in any category are constant. Some of the peaks and valleys observed are due to slight differences in the time at which the data were dumped. Some of them are caused by changes in calibration of the system. The majority of the variations are a reflection of variable traffic characteristics. However, there are a number of trends that are clearly demonstrated.

Growth Trends

An increase in weight for all heavy trucks (Types 11 through 19) is shown in Figure 11. This increase in weight of heavy trucks (also indicated by an increase in average ESAL in Figure 12) was partially due to a calibration change in early October of 1984. However, an increase in the weekly ESAL can be seen in Figures 17 through 19, which is due to more than the calibration increase. The actual increase in weekly ESALs is about 1,000 ESAL/yr (about 7 percent).

The volume of longer combination vehicles (Types 12 through 19) is increasing (see Figure 15); it is shown in Figure 16 that this is caused by an increase in Types 14, 15, and 17, which are predominantly doubles. Shown in Figures 17 and 18 are accompanying increases in ESALs, in particular from Types 14, 15, and 17. This trend is encouraging because the contribution of five-axle twins (Type 12) is decreasing, as verified in Figure 20, which also shows a decrease in ESAL contribution from Type 11 (3S-2). The five-axle twin truck is potentially the most damaging vehicle when fully loaded to the legal limit, because it has the minimum number of axles feasible, which are all single axles.

Seasonal Trends

It is shown in Figures 15 and 16 that there is a trend of higher truck traffic in summer and autumn. However, this is small compared with the total traffic stream data (Figure 14), which shows clearly that traffic is heaviest in August and lightest in January and February. The range for 1984–1986 is from about 80,000 vehicles/week to about 115,000/week (both lanes).

Truck Characteristics

The percentage of trucks in Lane 1 varies from about 15 to 21 percent (see Figure 21), and in Lane 2 from about 12 to 17 percent. There are about 14,000 trucks/week (in both lanes), on the average (see Figure 15), with a total ESAL of about 20,000 (see Figure 19). Type 11 trucks provide about 60 percent of the weekly ESAL (see Figure 20), but a little under 50 percent of the total truck volume (see Figure 15). As shown in Table 4 and Figure 12, the average ESAL values for Type 11 trucks are about 1.5 and for Type 12 trucks about 2.1. These values are high as a result of the WIM weights being about 5 percent higher than static weights, and if the weights were reduced by this amount, average values of about 1.2 and 1.7 would result for Type 11 and Type 12 trucks, respectively. These estimates are conservative because WIM weights were less than 5 percent high in Lane 2 (see Table 5).

The average weight of tridem axles has increased dramatically since the fall of 1985, as shown in Figure 13. Although the total number of tridem axles is very small (see Table 1), their frequency is increasing, and this is a trend to be observed carefully in the future.

Speed

It is shown in Figure 22 that after one initial calibration change, the average traffic speed has remained fairly constant—at about 59 mph. Cars are consistently faster than trucks by about 2 mph.

Summary

The data collected from a continuously operational fixed site such as the Jefferson site provides valuable information. However, the considerable data collected must be processed before they are usable. Once processed the data could be used to establish statistical sampling plans for future sites. For instance, it is possible to establish exact patterns for traffic flow by hour, day of the week, and week of the year. Thus, data collected for short periods from similar sites could be extrapolated with reasonable confidence. Such techniques could be used for simple traffic counts or for portable WIM sites.

CONCLUSIONS

The following conclusions are drawn from the study:

1. AVC systems provide accurate and continuous data about vehicle volume, classification, speed, and weight. These data need to be adjusted to account for differences in WIM weights and static weights and for those vehicles that are not classified or not weighed;
2. Procedures for processing the data from the Jefferson traffic-monitoring site have been presented. These present the data in easy-to-read plots and tables that show distinct trends in the data. Because adjustments have not yet been made to the plots, only general trends and average values are emphasized at this time;
3. A major trend observed is the seasonal variation in traffic volume, which shows that the weekly volume of cars and other light vehicles traveling in summer is about 25 percent more than in winter. Similarly, it has been found that traffic speeds are uniform, with average truck speeds about 2 mph less than car speeds;
4. Data obtained from the Jefferson WIM site can be conveniently used for establishing pavement design parameters and various other traffic parameters for a variety of design and planning activities;
5. An example of significant pavement design data is the definition of ESAL for each truck type. For example, the average ESAL for Type 11 (3S-2) trucks is about 1.5, and that for Type 12 (2-S1-2) about 2.1. These values would adjust to about 1.2 and 1.7, respectively, if the difference between WIM and static weights is considered; and
6. There is a trend of increasing volume of longer-combination vehicles (Types 12 through 19). Within this group, the volume of Types 14, 15, and 17 is increasing and the volume of five-axle twins (Type 12) decreasing. This should slow the rate of increase in ESALs, because the vehicle types that are increasing have more axles than those that are decreasing.

FUTURE DEVELOPMENTS

Data from a fixed WIM, such as those at the Jefferson site, are of limited usefulness as they can only be extrapolated to similar Interstate sites. Nevertheless, they provide insight into traffic behavior, insight that was impossible to obtain before reliable WIM data became available. OSHD is in the process of developing a long-range plan for making use of WIM and associated vehicle-monitoring technology. The plan will address the recommendations of the Federal Highway Administration, as published in the 1985 *Traffic Monitoring Guide (5)*. This will require use of at least two portable WIM devices to be used continuously at selected sites on a statewide basis. These would be used in conjunction with the bridge WIM.

Developments in AVI are anticipated, indicating that they may be used on a widespread basis in the near future. A new port of entry will become operational on I-5 southbound at Woodburn during 1986. This will have the capability of allowing trucks with AVI devices to bypass the static scales and PUC station provided that they meet both weight and PUC requirements. It is anticipated that WIM/AVC/AVI technology will play a vital role in highway research in the near future and that both public and private sectors will benefit.

ACKNOWLEDGMENTS

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Automated Acquisition of Truck Tire Pressure Data

WILEY D. CUNAGIN AND ALBERT B. GRUBBS

Recent field studies have established that operational truck tire inflation pressures are much higher than those typically assumed in the pavement design process. Field data have shown that tire inflation pressures for trucks operating on the highway average between 95 and 100 lb/in.^{2} (psi), whereas 75 to 80 psi is usually assumed in pavement design. Other work has shown that these tire pressures are not uniformly distributed across the area of contact between the tire and the road surface. One of these studies indicated that contact pressures at the outer edge of the contact area can be as high as twice the tire inflation pressure. This situation is suspected of causing significant levels of premature failure in pavement structures in Texas. Presented in this paper are the results of a study into the feasibility of automatically monitoring the contact tire pressures produced by trucks while they are in motion by monitoring tire footprint dimensions and weight. The work undertaken has included *(a)* A review of principles of tire contact pressure measurement and available sensor technology; *(b)* An assessment of the feasibility for using each principle/technology for truck contact pressure measurement; and *(c)* development of the concept for an independent tire contact pressure measurement system, as well as options for incorporating an automatic contact tire pressure-sensing feature into current operational truck weigh-in-motion systems. Described in this paper are the results of work performed by the Texas Transportation Institute, sponsored by the Texas State Department of Highways and Public Transportation (hereinafter referred to as the Department) in cooperation with the Federal Highway Administration of the U.S. Department of Transportation.

Recent field studies indicate that operational truck tire inflation pressures are higher than normally assumed. Therefore a concept has been developed for measuring tire contact pressures as part of a weigh-in-motion system.

TIRE CONTACT PRESSURE MEASUREMENT

Tire contact pressure is the pressure on the surface of the pavement at the tire/surface interface. This value depends on both the applied load and the area to which it is applied. The acquisition of tire contact pressure data requires either direct or indirect measurement of the load on each tire and the lateral and longitudinal dimensions of the contact area.

There are basically two approaches available for automatically acquiring truck tire contact pressures. One of these is to provide the load input from conventional truck weigh-in-motion (WIM) equipment and to obtain truck tire footprint information from one or more other sensors. Another approach is to design a sensor that directly measures the tire contact pressure.

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TIRE CONTACT AREA MEASUREMENT

No method currently exists for directly measuring the tire contact area of a moving vehicle. Therefore, the computation of the tire contact area is necessarily approximate, because the shape of the tire footprint is neither regular nor constant. As shown in Figure 1, it is nearly circular at the leading and trailing edges and reasonably straight on the sides. As tire inflation pressure increases, the tire footprint becomes smaller, less rectangular, and more circular. Three methods are presented in this section for estimating tire contact area by measuring selected characteristics of the tire footprint. These are: axle sensor arrays; axle sensors in combination with WIM sensors; and pressure-sensitive devices. (This latter technology has been applied to discriminating between single and dual tires on the same end of an axle by at least one vendor of WIM systems, but their product has been discontinued and was not appropriate to the measurement of the tire contact area.)

Use of Axle Sensors for Tire Contact Area Measurement

One way to measure the tire contact area is to detect the edges of the tire footprint to provide values for the geometric parameters of the shape of the footprint. For example, if the footprint were rectangular, it would be necessary only to detect the leading, trailing, and side edges of the footprint according to the following idealized procedure.

Three axle sensors are used in the configuration shown in Figure 2. Two are placed laterally in the traffic lane, perpendicular to the direction of travel. The third is placed diagonally, at an angle of 45° to the direction of travel. Vehicle speed measurement is provided by the two lateral axle detectors. The length of the tire footprint is measured by the first lateral axle sensor. The diagonal axle sensor aids in computing the width of the tire footprint.

Figure 3 shows an example of the tire contact area measurement process. Vehicle speed is obtained from the actuation times of the two lateral axle detectors as follows. At time T_1, the leading edge of the tire strikes the first lateral axle sensor, producing the leading edge of an electrical pulse from electronic detection circuitry connected to the sensor. The electrical pulse stays up until time T_2, when the trailing edge of the tire leaves the first lateral axle sensor. At time T_3, the outer leading edge of the tire makes contact with the diagonal axle sensor, resulting in an actuation and producing the leading edge of a second electrical pulse. This electrical pulse stays up until time T_4, when the inner trailing edge of the tire leaves the diagonal

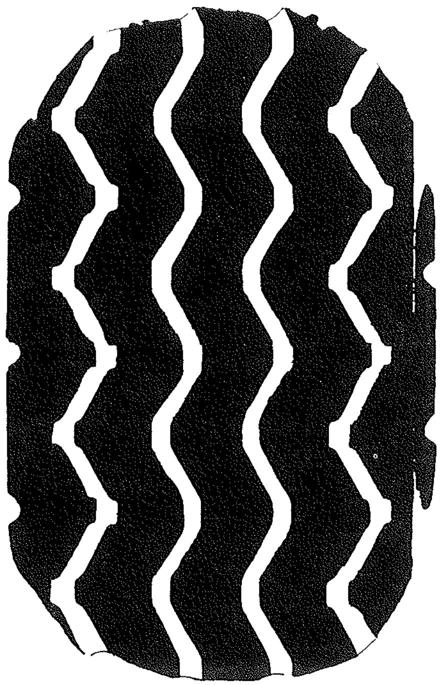
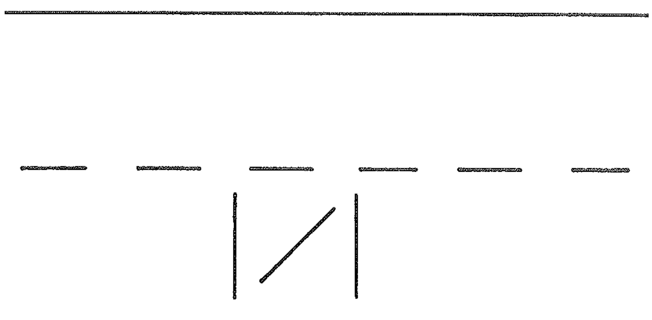


FIGURE 1 Typical tire footprint.



Z - configuration

FIGURE 2 Axle sensor configuration.

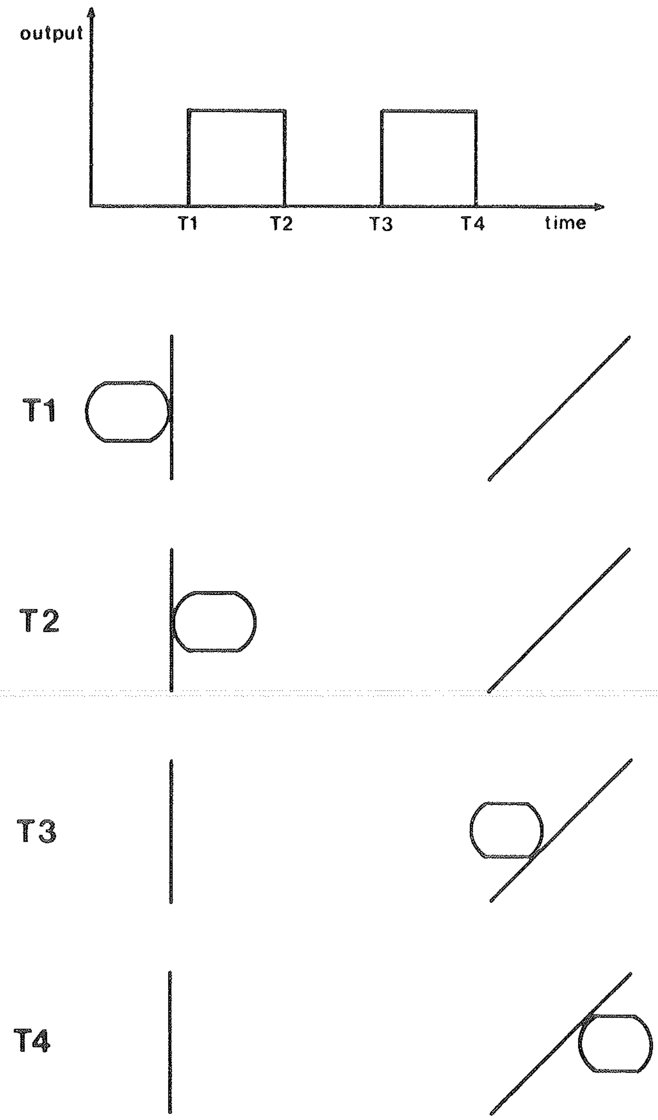


FIGURE 3 Axle sensor operation.

axle sensor and the electrical pulse drops to zero. At time T_5, the leading edge of the tire strikes the second lateral axle sensor, producing the leading edge of another electrical pulse. The electrical pulse stays up until time T_6, when the trailing edge of the tire leaves the second lateral axle sensor. The times at which the leading edge of the tire strikes the two lateral axle sensors are used to compute vehicle speed according to the following equation:

$$\text{Speed} = D/(T_5 - T_I) \tag{1}$$

where speed is in inches per second, actuation times T_i are in seconds, and D is the distance in inches between the perpendicular axle detectors. The footprint length is then calculated from the following equation:

$$\text{Length} = (T_2 - T_1) \times \text{speed} \tag{2}$$

where length is in inches.

Assuming the diagonal axle sensor is at an angle of 45°, the width of the hypothetical rectangular tire footprint is then calculated with the following equation:

$$\text{Width} = [(T_4 - T_3) - (T_2 - T_1)] \times \text{speed}/1.414 \tag{3}$$

where width is in inches. The constant 1.414 is a correction factor for the 45° angle.

Combining Equations 1 through 3 provides an equation for computation of the tire contact area from the actuation times of the sensor configuration shown in Figure 2.

$$\begin{aligned}\text{Area} &= (T_2 - T_1) \times [(T_4 - T_3) - (T_2 - T_1)] \times \text{speed}^2 \\ &= (T_2 - T_1) \times [(T_4 - T_3) - (T_2 - T_1)] \\ &\quad \times [D/(T_5 - T_1)]^2\end{aligned} \tag{4}$$

where the area is the tire footprint contact area in square inches. If the weight of a wheel is known, then the average tire contact pressure over the tire footprint is given by

$$\text{Pressure} = \text{weight/area}$$

where the tire contact pressure is psi, the weight of the tire is in pounds, and the area is in square inches.

Equipment for obtaining the wheel weight is discussed in the next section. With regard to measuring the tire contact area

using the procedure described above, it is useful to consider the assumptions on which the derivation is based and the impacts of deviations from the assumed conditions on the measurements. The assumptions made were that

1. The tire is rectangular,
2. The tire contact area is constant,
3. The speed through the sensor area is constant,
4. The distances between sensors are constant, and
5. The actuation times are precisely recorded.

Each of these assumptions will be examined in the following paragraphs.

The accuracy of automatic measurement of tire contact area using axle detectors is dependent on the assumed shape of the tire footprint. It is not necessary that the assumption of a rectangular area be met, but it is necessary that algorithms be developed that can relate the actual tire contact area to the actuation times recorded.

Tire footprints are generally rectangular with circular leading and trailing edges and straight side edges when inflated at approximately 80 psi. However, as inflation pressure is increased, the footprint becomes smaller and more circular for the same load. As this occurs, the axle sensor configuration of Figure 2 will not work as intended. The peak on the leading and trailing edges of the tire will cause the measurement system to overestimate the effective length of the tire footprint. Conversely, the shape of the same leading and trailing edges may result in an underestimate of the tire width because the front edge of the tire rather than the outer edge may be activating the diagonal sensor. This is particularly true for diagonal sensors, which make an angle of less than 45° with the lateral dimension of the roadway. However, it appears that the length and width measurement errors are offsetting. The degree to which this circumstance holds in actual field conditions needs to be evaluated.

The impact of tire shape on tire contact area measurement with axle sensors can be considered by analyzing Figure 4, which is based on Figure 1 and an actual tire footprint. The actual area of the footprint is 66.97 in.^{2}. The maximum length of the pattern is 10.84 in. and the maximum width is 6.95 in. The inflation pressure is 80 psi. Assuming that the truck with this tire is travelling at 55 mph and it strikes the first lateral axle sensor at time $T_1 = 0$, the following times will be recorded:

$T_1 = 0.000$ sec
$T_2 = 0.011$ sec
$T_3 = 0.012$ sec
$T_4 = 0.032$ sec
$T_5 = 0.198$ sec
$T_6 = 0.209$ sec

Then, from Equations 1 through 3:

$$\begin{aligned} \text{Speed} &= 192 \text{ in.}/0.198 \text{ sec} = 969.697 \text{ in./sec} \\ &\quad (= 55.00 \text{ mph}) \\ \text{Length} &= 0.011 \text{ sec} \times 969.697 \text{ in./sec} = 10.67 \text{ in.} \\ \text{Width} &= [(0.032 - 0.012 \text{ sec}) - 0.011 \text{ sec}] \times \\ &\quad 969.697 \text{ in./sec } /1.414 = 6.17 \text{ in.} \end{aligned}$$

The measured contact area is then

$$\text{Area} = 10.67 \text{ in.} \times 6.17 \text{ in.} = 65.83 \text{ in.}^2$$

This value compares favorably with the 66.97 in.^{2} in the actual area of the tire footprint. However, it remains to be seen whether such results can be obtained from a full range of tire types, sizes, tread conditions, and inflation pressures.

Laboratory studies of the variations of tire footprint shape and area under load have produced results that indicate that as the load increases, so does the area of the tire contact footprint. The rate of increase is dependent on tire construction and the tire inflation pressure. Figure 5 reproduces plots of gross tire contact area versus load for different inflation pressures for one common type of truck tire. Figure 6 shows plots of gross tire contact area for radial (I and II) and bias (VII and VIII) tires as a function of load. Both Figure 5 and Figure 6 indicate that gross contact area is a linear function of load for the reasonable range of loading.

Laboratory and field studies are needed to determine the exact relationships among tire contact area, effective tire width, effective tire length, tire inflation pressure, and tire loading for moving trucks. As indicated in the previous paragraph, a considerable body of laboratory data now exists, so that extensive laboratory work will not be necessary for the development of operational automated tire contact area measurement techniques and devices. However, test track and field test runs will probably be needed to provide the required information.

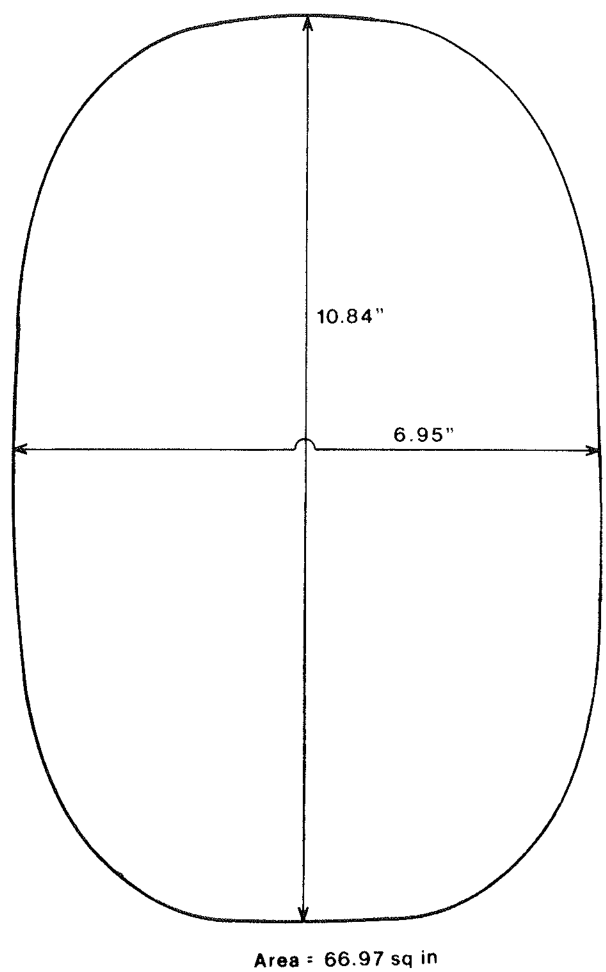


FIGURE 4 Tire footprint dimensions.

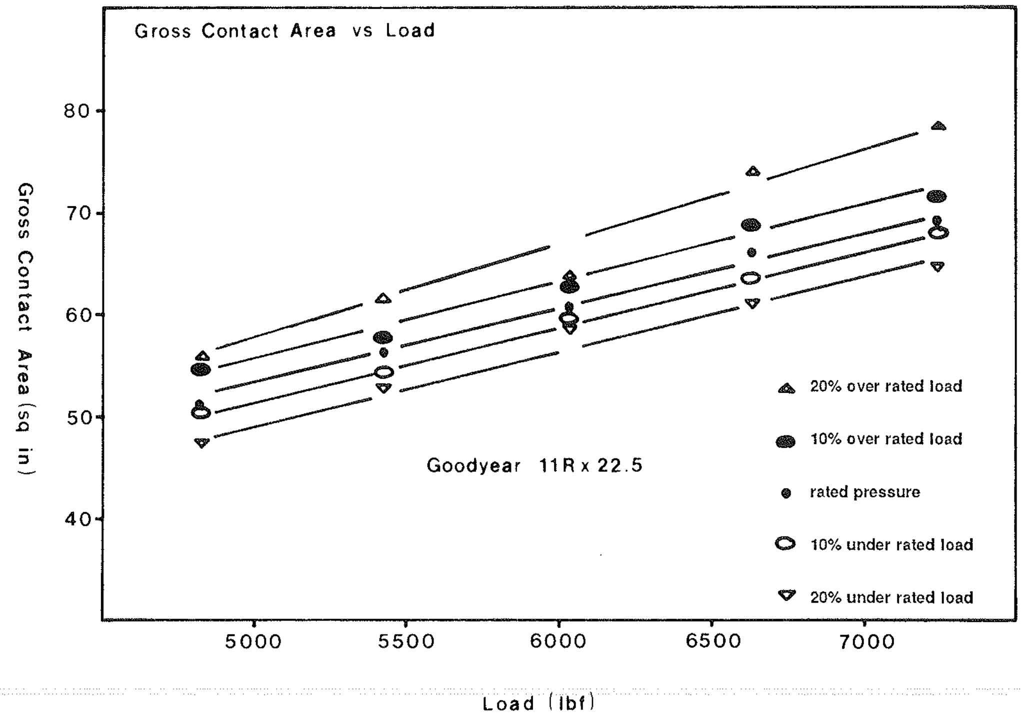


FIGURE 5 Gross tire contact area versus load and inflation tire pressure *(1)*.

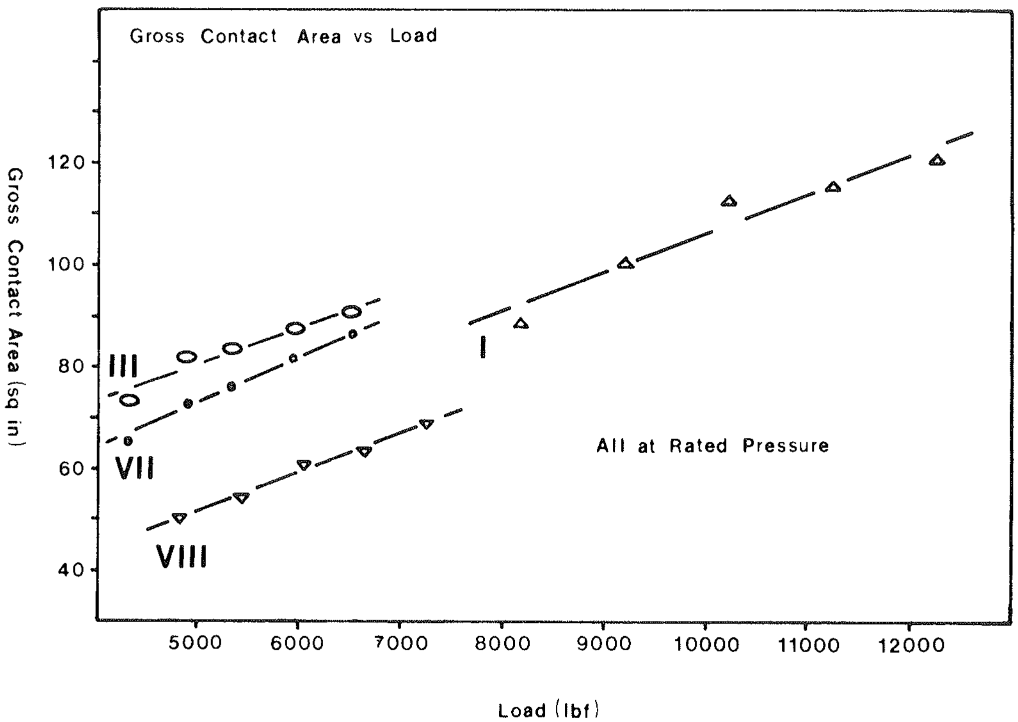


FIGURE 6 Gross tire contact area versus load for different tires *(1)*.

The assumption of constant vehicle speed is reasonable if the total length of the sensor array is less than approximately 30 ft and there are no conditions that are likely to cause vehicles to brake or accelerate. For this reason, monitoring sites should be chosen so that conflicts with other traffic are minimized and the variable speeds found in traffic congestion are avoided.

The distances between sensors are required for computation of speeds and subsequently tire contact area. An error of 6 in. in an assumed distance of 16 ft (a commonly used distance for speed measurement) for a vehicle traveling 55 mph can result in a 1.59 percent speed error and a 3 percent error in calculating the tire contact area for rectangular tire footprints. This error rate alone is not significant, but because it can be added to other sources of error it is clear that care must be taken in sensor placement.

Nearly all automatic electronic traffic data-collection systems acquire data digitally. That is, the electronic system controller checks the status of each axle detector at preset intervals

to determine if it has been actuated. This interval is usually approximately 1 ms. This use of periodic sampling limits the accuracy of speed and therefore tire-dimension measurements. For example, assuming a sensor spacing of 16 ft, an actual vehicle speed of 55 mph, and a sampling interval of 1 ms, an error of ½ ms in the actuation time for each axle sensor is expected. This would result in a 1 percent error in the measurement of tire contact area due solely to this source. Again, this error is not serious alone but it can contribute to unacceptable performance. The measurement of tire contact area requires as much precision as possible within the constraints of budget and technology.

Use of a Combination of Axle and WIM Sensors for Tire Contact Area Measurement

The simplest method for the addition of a tire contact area measurement feature to an existing WIM system is shown conceptually in Figure 7. The Radian WIM system sensor configuration used by the Department was chosen for illustration. The diagonal line shown in Figure 7 is an axle sensor. Vehicle speed measurement is provided by the Radian system using inductive loops. The weight transducer acts as both a weight and an axle sensor for the measurement of the length of the tire footprint. The diagonal axle sensor aids in computing the width of the tire.

Figure 8 shows an example of the tire contact area measurement process. Vehicle speed is obtained from the WIM system using inductive loops. At time T_1 the leading edge of the tire strikes the WIM sensor, producing the leading edge of an electrical pulse of the weight signal. The electrical pulse stays up until time T_2, when the leading edge of the tire begins to leave the WIM sensor. At time T_3 the tire has left the WIM sensor entirely and the electrical pulse drops to zero. At time T_4 the tire footprint makes contact with the diagonal axle sensor, resulting in an actuation and producing the leading edge of a second electrical pulse. This electrical pulse stays up until time T_5, when the inner trailing edge of the tire footprint leaves the diagonal axle sensor and the electrical pulse drops to zero.

Given the vehicle speed and the axle sensor actuation times, software can be developed to compute the approximate tire contact area using the following equation:

$$\text{Area} = (T_3 - T_2) \times [(T_5 - T_4) - (T_3 - T_2)] \times \text{speed}^2$$

where the T_i are in seconds and the speed is in inches per second.

Use of Pressure-Sensitive Mats for Tire Contact Area Measurement

Several types of pressure-sensitive material exist that could be used for the automatic measurement of tire contact area. These sensors operate on the principle that the area of the mat over which the tire passes can be automatically identified. This objective can be accomplished in several ways. Two of these are: *(a)* the fabrication of a pattern of pressure-sensitive elements as indicated in Figure 9; and *(b)* the recording of the pattern of activation of a solid pressure-sensitive sheet. Although these approaches have not yet been applied to the measurement of tire contact area (or pressure), they have been used for several years in commercial and industrial applications. The most commonly used material for this purpose is

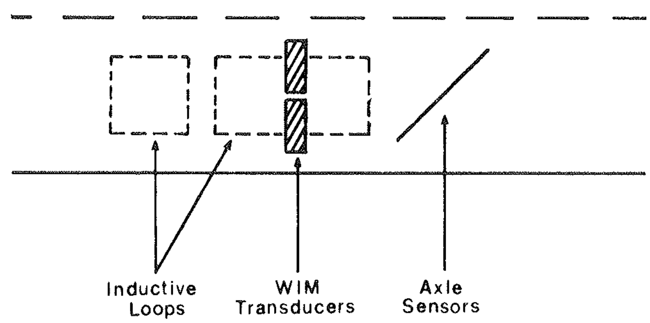


FIGURE 7 Axle sensor configuration for modified Radian WIM installation.

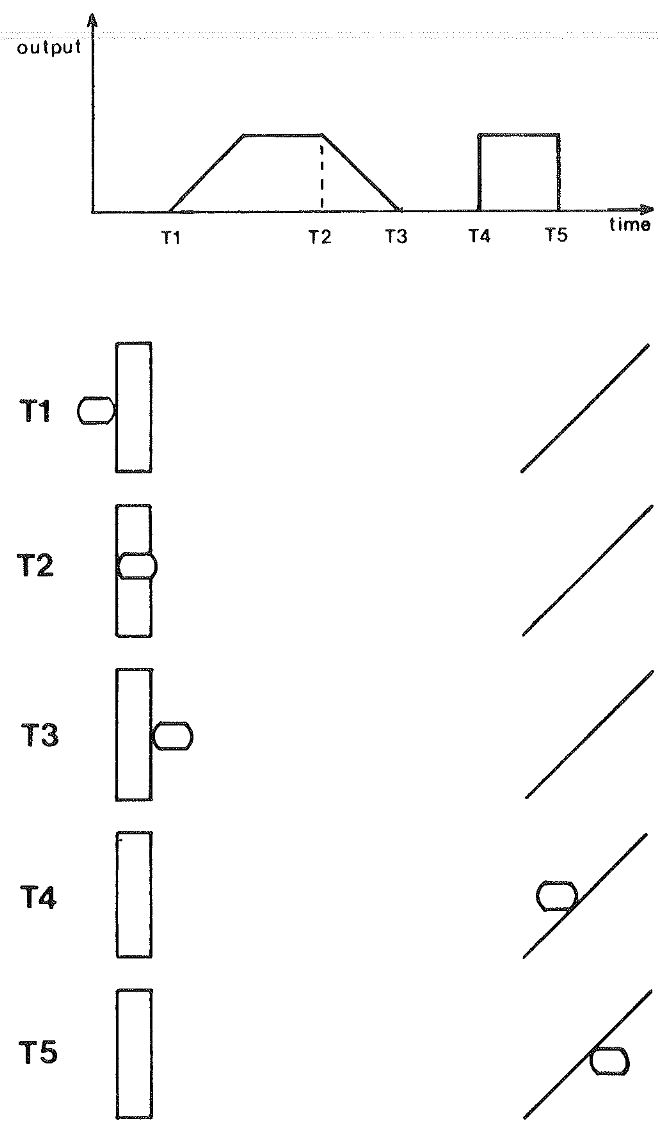


FIGURE 8 WIM/axle sensor operation.

piezoelectric film. It is discussed briefly in the section dealing with specific sensor technologies.

The pattern of pressure-sensitive elements shown in Figure 9 is used to detect the tire contact area by determining if there is a load on each element during a single sampling cycle of the digital electronic traffic-monitoring system. Each element covers a known surface area, so that the total contact area can be calculated by adding the areas of the actuated sensing elements. For example, consider Figure 10, which shows the tire footprint of Figure 1 superimposed on the pattern of pressure-sensitive elements from Figure 9. The tire actuates 5 percent of the elements on the mat, so it is assumed to have an area that is 5 percent of the 1,296 in.^{2} mat area, or 64.8 in.^{2}. This compares to the 66.97 in.^{2} area that was actually measured.

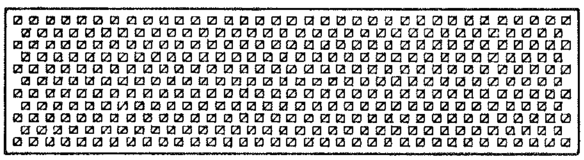


FIGURE 9 Pattern of pressure-sensitive elements.

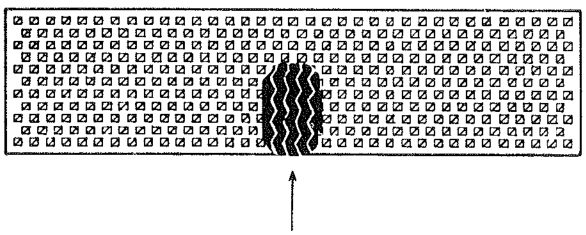


FIGURE 10 Operation of pattern of pressure-sensitive elements.

Clearly the density of the elements has a direct effect on the accuracy of the measurements. The more elements that are placed in a given area, the more accurate will be the estimate of the tire contact area. In order to provide an accuracy of ± 5 percent, it will be necessary to provide one sensor element for each in.^{2} of the mat surface area.

This type of sensor also obviously has a possible application to direct measurement of tire contact pressure. This approach is discussed in a later section.

Another possible approach is to construct a composite mat consisting of two sheets, each of which contains strips of pressure-sensitive material. As shown in Figure 11, the first sheet would have the strips oriented to correspond with the longitudinal direction of the traffic lane; the second sheet would have the strips oriented laterally so that they are perpendicular to the strips on the other sheet. As illustrated in Figure 12, when the tire footprint is covering any portion of a strip it is detected. The coordinates of the intersections of actuated longitudinal and lateral strips can then be determined. If there are a total of 324 intersections of longitudinal and horizontal strips covering 1,296 in.^{2} and 17 of these are actuated, then the calculated tire contact area is 68 in.^{2}. As with the use of the pattern of individual pressure elements, the density of the strips has a direct effect upon the accuracy of tire contact pressure.

The third application of pressure-sensitive material to the measurement of tire contact area is to use a single solid sheet,

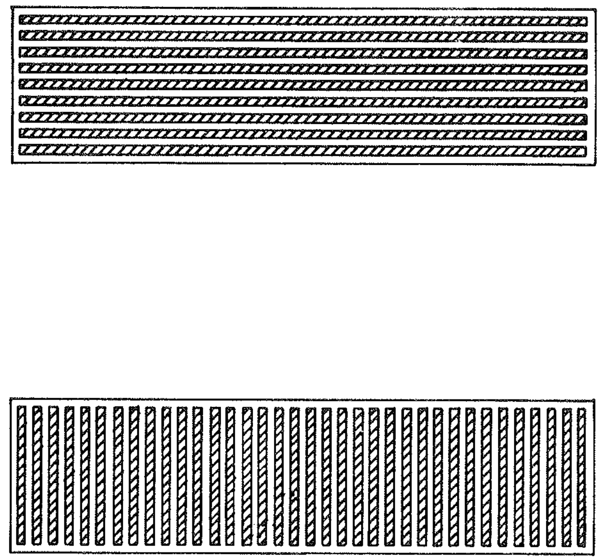


FIGURE 11 Pattern of pressure-sensitive strips.

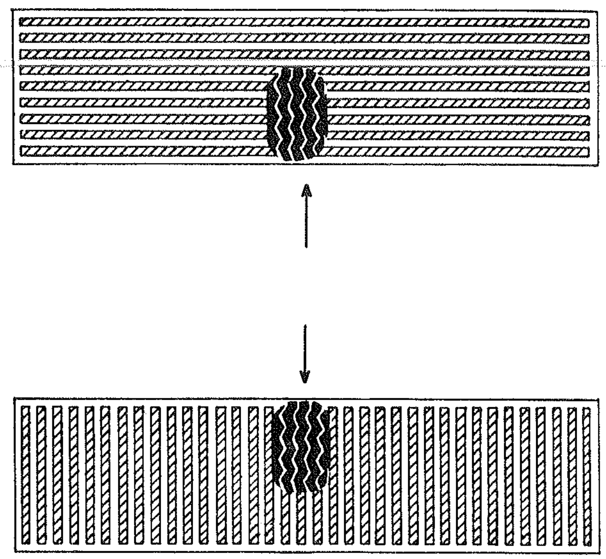


FIGURE 12 Operation of pattern of pressure-sensitive strips.

taking advantage of the time it takes for the actuation response of an area of the material to reach the electronic detection circuitry. When the tire is on a specific area, an excess electrical charge is generated in the crystalline structure of the pressure-sensitive material. This excess charge travels through the material at a known speed. There are receptors along the edges of the mat that receive these generated signals and interpret the times they are received as actuations of the surface contact area. Although a moving tire is a more complex phenomenon than others that this approach has been used to address in commercial, industrial, and military applications so far, algorithms could be developed for its successful application to tire contact area measurement.

The three configurations of pressure-sensitive material already described have been discussed in the context of a mat. However, it is also possible that a narrow strip could detect tire length in a manner similar to an axle detector while it sensed tire width as a pressure sensor.

DUAL TIRES

The preceding discussion has applied to measuring the tire contact area of a single tire. However, it is most common that heavy trucks have dual tires on each side of an axle (except for the steering axle). The use of the diagonal axle sensor will require that statistical relationships be developed between the apparent width produced by two tires together, which appear to be one to the sensor and the actual contact area of the pair of tires. Alternatively, if a piezoelectric cable is used as the diagonal axle sensor, the width of each tire can be determined from the difference in the magnitude of the cable output produced by the number of tires. The discussion of piezoelectric cable in the following section on axle sensors includes a figure that clarifies the operation of this sensor for discriminating between one and two axles.

Any of the three pressure-sensitive material sensor configurations are capable of determining if an actuation is caused by one or two axles. This is due to the fact that each has the ability to monitor what is happening in each section of the mat at any time.

APPLICABLE SENSOR TECHNOLOGIES

This section presents a discussion of vehicle sensor technologies that are applicable to the objective of automatically measuring tire contact pressures. Also included is a description of other technologies not now in use for vehicle sensing that could be applied to this problem.

Axle Sensors

Pneumatic Tubes

Pneumatic tubes are easily the most widely used axle sensor. They operate by the rapid compression of a trapped volume of air in a section of tubing with the subsequent mechanical actuation of a diaphragm. The actuation of the diaphragm in turn produces an electrical pulse that is delivered to the recording circuitry. Pneumatic tubes are always installed in portable situations. These devices have several advantages for application to tire contact area measurement. They are inexpensive and can be easily installed by one person in less than 10 min under low traffic conditions. These sensors are both durable and reusable. The operational life is approximately 6 months under daily use and moderate traffic levels.

Pneumatic tubes also have some serious disadvantages that may limit their use as part of an automatic tire contact pressure-detection system. They are not well suited to high traffic volume conditions. Tests have shown that pneumatic tubes placed side-by-side in the same lane can produce counts that vary by as much as 30 percent over a 15-min time interval. The tubes also tend to work loose under heavy traffic, reducing the accuracy of the critical actuation time measurements needed to compute vehicle speed and tire contact area.

Tapeswitches

Tapeswitches are generally used in temporary applications but can be installed permanently. These sensors are basically long, narrow pairs of metallic contacts separated along their edges by insulation. The device is protected from environmental conditions by a waterproof vinyl sheath.

The cost of each tapeswitch ranges from approximately $30 to $100, depending on the type of construction. The less expensive type consists essentially of the sensing element with its protective covering. The more expensive type adds a rigid steel frame to provide durability.

Tapeswitches are usually affixed to the highway surface with adhesives. The adhesives that have been used for this purpose vary widely and include tape with adhesive material on two sides, cloth impregnated with rubberized asphalt, and masking tape.

The basic tapeswitches are less than $^3/_{16}$ in. high and are not particularly conspicuous to drivers. However, the addition of the steel frame increases the profile to $^7/_{16}$ in., which makes it much more conspicuous. They are more accurate than pneumatic tubes, due principally to the fact that they can be securely fastened to the surface of the roadway. The closure of the switch is also more reliable than the operation of the diaphragm in the pneumatic tube sensor.

Piezoelectric Cable

Piezoelectric cable operates on the principle that electrical charge is generated when certain crystalline materials are subjected to stress. One type commonly in use now is a coaxial cable with crystalline piezoelectric powder as the dielectric material. Both temporary and permanent installations of this sensor have been successfully made. As an axle sensor, piezoelectric cable has the advantage that it, like the tapeswitch, can be exactly placed to provide the level of accuracy for actuation measurements needed in determining tire contact pressure.

Piezoelectric cable also has one significant advantage over other axle sensors for use in measuring tire contact area. That is, the magnitude of the signal produced by the cable is proportional to the pressure applied to it. This sensor is therefore able to detect whether a single or dual tire has caused the actuation. For example, Figure 13 shows the general form for the signals produced by single and dual tires passing over a piezoelectric cable axle sensor placed diagonally across the right wheel path of a traffic lane.

Triboelectric Cable

Triboelectric cable is coaxial cable that produces an electrical charge on its conductive surface because of friction between its constituent materials when subjected to stress. Commonly available commercial and industrial types of coaxial cable exhibit this property when subjected to vibration or flexure. For permanent installations, the cable is usually encased in epoxy or polyurethane and placed in a slot in the pavement, which is then sealed. For temporary applications the cable is used similarly to the piezoelectric cable previously described. Its accuracy as an axle sensor is based on the accuracy with which it can be placed and maintained.

Capacitive

Capacitive axle sensors are based on the deflection of conducting surfaces caused by the passage of a wheel. The change in

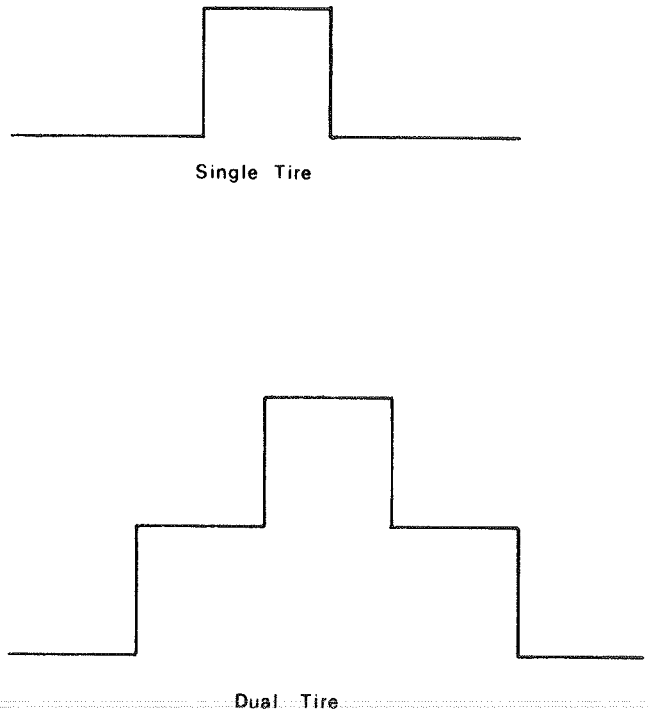


FIGURE 13 Axle detection signals produced by single and dual tires passing over piezoelectric cable.

separation of the conductive surfaces results in a change in the capacitance of the assembly. Coaxial cable can be used for this purpose in a manner similar to that described for the triboelectric cable. The principal difference in the two applications of the coaxial cable is in the signal-processing electronics.

Another form of capacitive axle sensor is the capacitive strip/mat sensor. This device has been studied and used extensively as a weight sensor and also might be used as an axle detector. However, it does not appear to offer any advantages over other technologies for this application.

WIM Systems

As indicated previously, it is necessary to acquire the weights of wheels while the truck to which they are attached moves over the tire contact pressure sensor array. A significant amount of effort has been expended and is now in progress to develop equipment and techniques for performing this function. The available commercial truck WIM equipment is described in the following paragraphs. An assessment of the likelihood of success of incorporating automatic tire contact pressure features into each WIM system is provided. In general, this feasibility depends on the ability of a system to accurately measure the wheel loads on one side of an axle.

Radian Corporation

The Radian Corporation WIM system has been in use by the Department since 1974. It consists of an independent weight transducer in each wheelpath. The weight sensors provide both weight and axle sensor functions. This system is appropriate for inclusion of a tire contact pressure feature in the configuration described in the previous section. That is, only a single diagonal axle sensor needs to be added to the sensor array (see Figure 7). The software could then be modified to make the necessary calculations and store the data. More detail about this process is included in the following sections.

Streeter Richardson

The Streeter Richardson Division of the Mangood Corporation produces two different types of WIM system: one permanent and one portable. The permanent unit has one weighing platform in each wheel path. Speed data are provided by two inductive loops. Both truck weight and axle-sensing functions are provided by the weighing platform. This equipment is adaptable to truck-tire contact pressure measurement by the addition of a diagonal axle sensor in exactly the same way as that described for the Radian system, shown in Figure 7. This equipment is not easily moved between sites and generally stays where it is installed.

The Streeter Richardson portable WIM system uses a capacitive weighmat with two inductive loops for measuring speed. This equipment is also usable for measuring truck tire contact pressure by the addition of a diagonal axle sensor following modification similar to that described for the Radian WIM system. However, there is only one weight sensor and it is in the right wheel path. Consequently, only tire contact pressures for the right side tires can be obtained.

International Road Dynamics

Conceptually, the IRD WIM system is similar to both the Radian and Streeter Richardson permanent systems in that there is a weight sensor in each wheel path that also serves as an axle sensor for computing the distance between axles. Speed information is provided by inductive loops. Adaptation of this equipment to tire contact pressure measurement can be accomplished by the addition of a diagonal axle sensor, as shown in Figure 7, and software to process the data.

Golden River Corporation

The Golden River Corporation markets a portable WIM system that uses the same sensor configuration as the Streeter Richardson portable WIM equipment. The principal difference in the two systems is that Streeter Richardson uses a Compaq portable microcomputer as the central processing component, whereas Golden River uses a dedicated microprocessor-based data collection system. Modification of this equipment to measure tire contact pressure is feasible in a manner similar to the Streeter Richardson portable system.

Siemens-Allis (PAT)

The Siemens-Allis (PAT) WIM system has wheel load sensors in each wheel path. Speed measurement is provided either by inductive loops or by two wheel load weighers, longitudinally spaced. This equipment can be modified for measuring tire contact pressure by the adjustments previously described for the other permanently installed WIM systems.

Bridge Weighing Systems

The Bridge Weighing System does not weigh individual wheel or axles directly. The wheel loads must be mathematically derived and users have had difficulty in obtaining acceptable levels of wheel weight accuracy with this equipment. This WIM system does not seem appropriate for measuring tire contact pressure in its current state.

Weighwrite

The WIM system offered by the Weighwrite Company operates only at very low speeds and weighs all tires on an axle simultaneously. Consequently, individual wheel load data are not available and this equipment is not appropriate for measuring tire contact pressure.

TIRE CONTACT PRESSURE MEASUREMENT SYSTEMS

The following discussion presents two conceptual configurations for measuring tire contact pressure automatically.

Option 1: Existing WIM Equipment with Tire Footprint Measurement

WIM equipment is widely used in the United States and is being implemented on an increasingly larger scale. It is therefore very attractive to use these existing devices for the measurement of tire contact pressure. Given that the WIM equipment will produce wheel loads, this option will require the addition of tire contact area sensors with a means of combining the WIM data with the tire contact area data for the calculation of tire contact tire pressure.

The department now uses two different WIM systems for the collection of truck weight data. The first of these is the Radian Corporation WIM system, which uses weight sensors incorporating strain-gauge load cells in combination with an IBM-XT microcomputer equipped with interface and signal processing electronics. The weight sensors are placed in prepared shallow excavations for each session of truck weighing. The microcomputer is housed within a van that has been modified for on-site truck weighing. In addition to the truck weight sensors, a pair of inductive loops is provided for each lane. These loops provide presence signals that are used for calculating speed and for activating the electronic subsystems of the truck weighing system.

The use of the IBM-XT microcomputer within this system contributes to the ease of this modification, because this equipment is generally easier to work with than the proprietary dedicated electronics formerly used in the system. As indicated in the previous discussion and in Figure 7, the only modification needed for the existing Radian WIM sensor array is a diagonal axle sensor. This device provides information about sensor width that is used with data already acquired by the WIM system (wheel weight and footprint length) to calculate tire contact pressure according to the relationship

$$P = W/A$$

where

P = the tire contact pressure in psi,
W = the wheel weight in pounds, and
A = the tire contact area in square inches.

As a part of this feasibility study, the Texas Transportation Institute (TTI) conducted limited tests to determine the best angle for the diagonal cable. The results of this activity showed that angles of less than 30° with the lateral dimension of the traffic lane could result in interference with the measuring process because of the nearly simultaneous production of signals by tires on opposite ends of the same axle. Angles of greater than 60° are difficult to use when the tires on both ends of an axle need to be measured. Limited tests of piezoelectric cable and other axle sensors confirmed that the piezoelectric cable should be considered for use as the diagonal axle sensor because of its ability to distinguish between single and dual tires on a wheel. The software required for this task can be obtained either by modifying the existing WIM system software so that the appropriate calculations can be made and the data elements (P and A) are stored with each record as it is acquired; or modifying the software used to process the data after they are collected.

Either approach will produce the needed results. The choice of the approach will depend on the availability of personnel familiar with the WIM system software and the economic constraints. The modification of the WIM system has, however, proven to be difficult in the past because of the reluctance of vendors to provide the source code and support required for the user or a hired consultant to effectively perform this task. At the same time, the vendors have generally had little interest in making changes to the software or hardware without significant charges. The best solution is therefore the second one, in which information output by the WIM system is used in conjunction with axle sensor actuation data to calculate tire contact pressures during the processing of the data at the end of a study—not during the data collection.

If the data necessary for calculating tire footprint length are not available but the outputs of vehicle speed and wheel load can be obtained from the WIM system, the configuration shown in Figure 2 can be used to acquire tire contact area for use in computing the tire contact pressure. Because the weight-sensor actuation data are not available in this case, it is unlikely that the software will be available for modification. It will therefore be necessary to compute tire contact pressure from the individual data elements at the data-processing stage.

Option 2: Direct Measurement of Tire Contact Pressure

As indicated earlier, pressure-sensitive materials exist that could be used in the development of a stand-alone tire contact pressure system without the addition of the output from a WIM system. Two approaches are described in this discussion. One uses piezoelectric film and the other uses piezoelectric cable.

As a part of this feasibility study and other research, TTI conducted limited tests on piezoelectric film to determine its suitability for use in a tire contact pressure detector. Previous research had focused on the use of the pyroelectric characteristics of the material as a passive infrared vehicle sensor. That work demonstrated that the material was very sensitive to

electrical noise and vibration so that it was not suitable as an infrared vehicle sensor without an extended research and development program. The application of the material to direct measurement of tire contact pressure presents similar problems that must be overcome. The direct pressure caused by the tire and sensed by the piezoelectric film must be isolated from electrical noise, vibration, and infrared energy. Fortunately, it is much easier to accomplish these tasks in the context of an enclosed mat than with an exposed element of film.

Another difficulty that must be overcome with the film is the lack of uniform sensitivity to pressure for the manufactured product. Areas of the same roll can vary by as much as 10 percent in sensitivity to pressure. The pattern of pressure-sensitive elements shown in Figure 9 was developed by TTI in order to offset this effect. By cutting the material into 1-in. squares, it is possible to measure the sensitivity of each square and sort the squares into groups of the same sensitivity. TTI has developed a method for testing large numbers of these elements quickly and efficiently. Squares with the same sensitivity are then used to make a tire contact pressure sensor using the pattern shown in Figure 9. In this application, the elements are each monitored to determine the magnitude of the signal being generated by tire contact pressure. This process can provide not only the average tire contact pressure but also a measure of the variation of the pressure across the tire contact area.

A microcomputer with interface and signal processing hardware and software are necessary to complete the system.

Research is now underway to assess the feasibility of using piezoelectric cable as a weight sensor. If this effort is successful, the resulting system could provide tire contact pressure data by installing three sections of the cable in a configuration similar to that shown in Figure 2.

CONCLUSIONS AND RECOMMENDATIONS

It is both feasible and desirable to acquire tire contact pressure data automatically. It appears that a quick implementation of this concept can be realized by adding one diagonal axle sensor to the Department's portable and permanent WIM systems. Both portable and permanent types of axle sensors are available. Direct measurement of truck tire contact pressure is also possible but will require additional research and development.

The availability of automatically acquired truck tire pressure data will allow for making policy decisions about the modification of the Department's pavement design procedures to take into account changing tire inflation pressures.

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Calibration and Accuracy Testing of Weigh-in-Motion Systems

PETER DAVIES AND FRASER SOMMERVILLE

Examined in this paper are the problems of calibration and accuracy testing for weigh-in-motion systems, and several approaches are proposed that might improve the present situation. After the definition of terminology, recent calibration techniques are outlined and a different approach is advocated, based on random vehicles. A statistical appraisal of the approach concludes the first part of the paper. In subsequent sections, a new technique is described for self-calibration of weigh-in-motion systems, based on the fact that loads on certain axles of particular truck classes show relatively little variation. Self-calibration offers the prospect of improved weigh-in-motion accuracy in between conventional calibration exercises. In the final sections of the paper, the questions of weigh-in-motion accuracy appraisal and weigh-in-motion performance standards are addressed. An accuracy "funnel" for the assessment of weigh-in-motion performance is proposed. This provides for an absolute accuracy tolerance at low axle weights, with a percentage tolerance at higher loads.

Weigh-in-motion (WIM) systems have been used for many years in the collection of axle load data and as a screening tool for the identification of potentially overweight vehicles. As the use of WIM systems has increased, a number of issues have emerged concerning the operation and evaluation of these systems. The authors of this paper set out to examine the fundamental problems associated with calibration and accuracy testing of WIM systems, and propose several approaches to improve the current situation.

Much of the content of the paper has been developed through a contract being undertaken by Castle Rock Consultants on behalf of the states of Iowa and Minnesota. This contract is for the development and evaluation of a low-cost automatic weight and classification system (AWACS) based on a piezoelectric axle load sensor. As part of this work, a test program was developed to statistically assess the accuracy of the system over a period of time.

Before aspects of calibration and accuracy testing are described, the commonly used terminology is outlined. A review of current calibration techniques has been included, together with a discussion of the potential for self-calibration of WIM systems. Following this, the main emphasis of the paper is on statistical techniques to assess the accuracy of WIM equipment. At the conclusion of the paper the problem of WIM performance specifications is examined, advocating a new approach to accuracy assessment.

TERMINOLOGY

The technique of in-motion weighing attempts to measure the mass of a vehicle, a wheel, an axle, or a group of axles on a vehicle by determining the constantly changing vertical dynamic force that is applied by the tires to the pavement surface. The vehicle mass remains constant, by definition, but the rolling wheel applies a dynamic force to the highway that might range from double the static weight, after the vehicle has traversed a bump, to zero when the tire bounces off the ground. This so-called dynamic weight of an axle or vehicle makes up a component caused by the static load of the vehicle and a superimposed dynamic force component, which results from the fluctuating motions imparted to the vehicle by external factors. The magnitude of the dynamic force component is dependent on vehicle, highway, and environmental characteristics *(1)*.

The accurate measurement of static loads for vehicles at rest is surprisingly difficult. In order to weigh the whole of a truck in one instant, scales have to be large and complex. Placement of the vehicle on the scale can affect scale response significantly. Worse, however, is the fact that most scales weigh trucks section by section; each time the truck moves forward its suspension system shifts and redistributes load between the axles. Leveling the pavement and releasing the truck's brakes can reduce these static weight measurement errors; significant errors, however, inevitably remain.

To infer accurate static axle or vehicle loads from dynamic measurements, the vertical acceleration of all the vehicle elements should ideally be zero. The sum of the vertical forces exerted on a smooth, level surface by the perfectly round and dynamically balanced rolling wheels of a vehicle at constant speed in a vacuum is theoretically equal to the static weight of the vehicle. This ideal situation is obviously impossible to attain in practice. In any event, external variables, such as vehicle, highway, and environmental characteristics, are only some of the factors that influence the WIM measurement. Internal errors associated with the measuring equipment can also contribute significantly to discrepancies between dynamic weights and their static equivalents.

In most cases, weigh in motion can only give an instantaneous indication of the axle or wheel load as the vehicle crosses the WIM scale. If the WIM system were to operate without measurement error, the measured load would be the true dynamic force exerted by the wheel at a particular point and time. Users of weigh-in-motion systems, however, frequently want to infer static weights from dynamic measurements. The actual difference between static and instantaneous dynamic weight is often treated as an integral part of the WIM error. In common usage, therefore, WIM errors are made up of three components, whose order of importance might be *(a)*

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actual static/dynamic force differences, *(b)* dynamic force measurement errors, and *(c)* static load measurement errors. It is generally impracticable to separate out these effects, and the remaining discussion treats these as a single, combined error or inaccuracy.

Terminology used to express so-called WIM accuracies varies. An impact factor (IF) can be calculated for axles or vehicles to give an indication of the system performance *(2)*

where

$$\mathrm{IF} = \frac{\text{WIM weight}}{\text{Static weight}}$$

Alternatively, WIM accuracies can be expressed in terms of static-to-dynamic weight differences using either absolute or percentage weight values. Absolute differences would be appropriate if weighing errors were approximately equal, irrespective of vehicle or axle weight. Percent differences are more appropriate if the size of the weighing error increases in proportion to the mass of the axle being weighed, which is more likely the case. If neither of these conditions is true, separate accuracies should perhaps be quoted for axles in different weight bands. The percentage difference (PD) and absolute difference (AD) can be defined by

$$\mathrm{PD} = \frac{\text{WIM weight} - \text{static weight}}{\text{Static weight}} \times 100\%$$

$$\mathrm{AD} = \text{WIM weight} - \text{static weight}$$

Whether IF, PD, or AD is used to express WIM accuracy, results for a single vehicle are of little interest in themselves. Random fluctuations may make static/dynamic differences (errors) substantial, or negligible, for any particular vehicle or axle. Accuracy assessment requires WIM system performance to be established over large samples of vehicles, for which a normal distribution of errors will commonly be observed. Performance across the sample is described by statistical measures associated with two further error parameters: systematic error and random error. Both kinds of error can be associated with internal measurement errors, or actual static/dynamic weight discrepancies resulting from external highway factors.

The systematic error is given by the mean of the error distribution for individual measurements, whereas the random error is measured by its standard deviation. Systematic errors can arise for reasons relating to the design, installation, or operation of the system and cause a repeatable bias in all measurements carried out at a particular time. Random errors, however, are uncontrollable and unpredictable, and are intrinsic to any measurement. The purpose of calibration is to compensate for systematic errors, reducing them as far as possible. The initial calibration may change over time, producing a varying systematic bias in the load measurements.

Current WIM system calibration techniques are briefly reviewed in the next section. The feasibility of a self-calibration technique is described in the subsequent section.

CALIBRATION TECHNIQUE

Current techniques for calibrating weigh-in-motion systems vary considerably among practitioners. Little research has been performed to develop a standard technique, with present practice appearing to have evolved from trial and error. A brief review of some of the different techniques adopted is presented below.

Hamrick describes the technique adopted by Idaho Department of Transportation to calibrate a bending plate WIM system *(3)*. Essentially, it involved using a three-axle test vehicle of 30,000 lb gross weight making a series of runs over the systems of 20, 40, and 60 mph. The system calibration was then adjusted to minimize the average differences between the dynamic and static gross weights. Repeat calibration exercises were undertaken approximately every 3 months but no data are available indicating the extent of the change in systematic error over time.

The approach used by the Minnesota Department of Transportation involved three types of test vehicle *(4)*. A two-axle, six-tire vehicle was used to estimate the variations between dynamic and static weights and allow a preliminary calibration to be made. Subsequently, three vehicles of different weights were run over the weight sensor at different speeds, with sample sizes ranging from 4 to more than 70. The systematic errors for each test vehicle ranged between −2.2 percent and +4.7 percent. No further adjustments were made to the calibration. However, in subsequent calibration exercises several months later it was reported that the systematic errors now lay in the range −16 percent to +30 percent. No firm conclusions were drawn from these results, but it was suspected that the equipment was faulty.

Chow performed an evaluation of the PAT and Streeter Amet WIM systems in 1982 *(5)*. As part of the evaluation, a two-stage calibration procedure was adopted: a preliminary calibration wherein a loaded truck was driven across the scales at different speeds, at least 15 times at each speed, and the calibration settings adjusted to minimize the systematic error; followed by a subsequent main calibration using random trucks with various axle combinations, suspensions, and loadings, crossing at various speeds. The whole calibration procedure, according to the report, was "laborious, tedious and difficult, and required about six days to accomplish."

An alternative calibration method developed by the Transport and Road Research Laboratory (TRRL) in the United Kingdom is described by Priest and Moore *(6)*. This particular technique involves the application of a static load, via a load cell, using a hydraulic jack acting against the underside of a specially reinforced heavy vehicle. Loads are applied to the center of the weigh scale through a rubber-faced mild steel disc, 8 in. in diameter. However, such static calibration can take no account of systematic errors resulting from dynamic effects such as pavement profile. The use of a mobile dynamic loading rig for WIM calibration has also been suggested, but this would suffer from the same limitation.

To summarize, the use of test vehicles for calibration is not wholly satisfactory because the effect of factors such as suspension type is not taken into account. Mobile loading rigs take no account of pavement profile. The best approach to calibration, while costly, would seem to involve the use of random vehicles selected from the population of vehicles to be weighed at a particular site.

INITIAL CALIBRATION

A perfect calibration would eliminate all systematic error in weight measurement for the population of trucks at that site. It is not practicable to measure the mean of the population error distribution directly, because it is impossible to measure every vehicle at a site. However, this value may be estimated statistically using a random sample from the vehicle population of interest.

The systematic error for the calibration sample can be eliminated by setting the equipment so that the sample has zero mean error. In practice, this may be derived by plotting the WIM output against static load for the sample and fitting the best straight line through the points. The sample size for initial calibration depends on the calibration accuracy required and the inherent variability of the data.

Previous experience indicates that the standard deviation (SD) of the PD distribution will be around 10 percent. The standard error of the mean (SE_m) is given by

$$SE_m = \frac{SD}{(n)^{1/2}}$$

where n is the number of static/dynamic weight comparisons. Confidence limits of 95 percent are given by approximately ±2 SE_m. Therefore, for a calibration accurate to ±1 percent, with 95 percent confidence, we require

$$n = \left(\frac{10}{0.5}\right)^2 = 400 \text{ observations}$$

As each observation comprises one single, tandem, or triple axle static/dynamic comparison, approximately 150 trucks will be required to achieve the accuracy stated. Clearly, the sample size will vary for different confidence and accuracy levels.

SELF-CALIBRATION

Self-calibration, or automatic calibration, is an approach that might allow the systematic error to be minimized by continuously monitoring any change in the calibration. Self-calibration features might play a major role in reducing the operational costs currently associated with the above method or other conventional methods of WIM calibration.

The principle of the approach is that the loads on certain axles of specific truck classes show relatively little variation, regardless of the loading condition of the truck. Therefore, provided that the WIM system includes automatic vehicle classification, a data base can be built up by the system consisting of axle load measurements for these particular axles. If the mean weight of these axles is calculated after the addition of each new set of measurements to the data base, the system calibration factor can be adjusted to force the mean weight to agree with a known long-term population mean.

One particular axle category has been suggested as suitable for potential use as the basis for a calibration feature *(7)* (C. Dahlin, in a letter addressed to Perry Kent, FHWA, 1983) and *(8)*. This is the steering axle of 3S2 trucks. The vehicle dimensions of 3S2s are such that the kingpin is usually located close to the center of the first tandem axle. Therefore, the loading on the vehicle has relatively little effect on the steering axle load.

To test this hypothesis, a data base of U.S. vehicle dimensions and weights, collected during a biennial Truck Weight Study (TWS) in Arizona, was analyzed by the authors. For the self-calibration technique to be practical, only classes of vehicle that commonly occur in the normal traffic stream can be used, and so for this analysis only 3S2s were selected. Of the 1,500 vehicle entries in the data base, over one-third were of the class being analyzed. The vehicle weights in the data base had been obtained from portable static weighing scales, accurate to ±2 percent, and collected during a period when no enforcement weighing was in progress.

A computer program was written to analyze the data base by first selecting only the vehicle type under consideration and then examining the axle weight of the steering axle. Each of the weights examined was sorted into classes of 400 lb in the range 6,400 lb to 14,000 lb (Figure 1). In total, 512 vehicles were used in the analysis, having a mean axle weight of 9,950 lb, with an associated standard deviation of 1,126 lb.

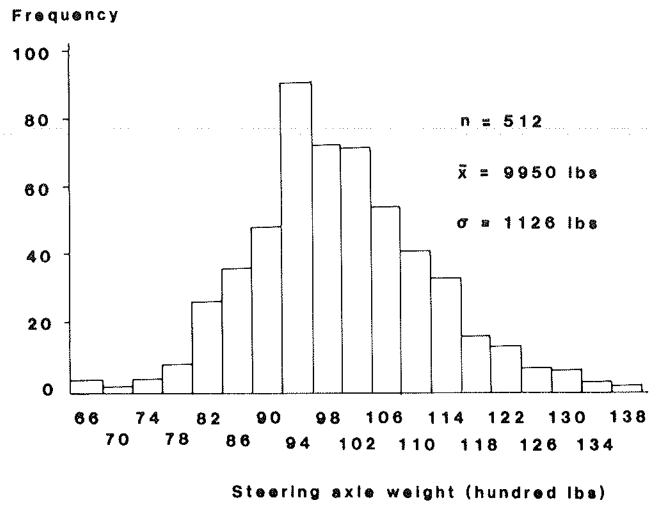

FIGURE 1 Analyzed 3S2 truck data.

If the assumption is made that the standard deviation of the sample is representative of the population as a whole, it is possible to determine the sample size required to give a specified standard error of the mean for a given confidence level. For example, for 95 percent confidence limits of ±100 lb in 10,000 or approximately ±1 percent, the required sample is given by

$$SE_m \text{ (standard error of mean)} = \frac{\sigma}{(n)^{1/2}}$$

where

σ = standard deviation and
n = sample size.

95 percent confidence = 1.96 SE_m (normally taken as 2 SE_m)

Therefore

$$1.96 \times 50 = \frac{1{,}126}{(n)^{1/2}}$$

where

$n = 132$

From the analysis of the limited data available, it would appear that the use of steering axle weight data from 3S2s for self-calibration purposes would be a practical proposition, although a number of issues would first need to be resolved. In particular, the reliability of the data used to determine the population mean and standard deviation would need to be assessed. It is possible that the data employed for this analysis are biased, caused by a proportion of illegally overloaded vehicles bypassing the weighing location even though no citations were being brought. These data may also be site dependent and time dependent as the construction and use of the vehicles change. This could be readily investigated given appropriate data for different sites and years, but would need to be monitored in the future. The accuracy of the vehicle classification, particularly for the class used in this technique, is a further consideration for the method to be effective and would need careful examination.

Because of the problems above, the technique would not totally eliminate the need for individual site calibration on a regular basis but could be used in several different ways to improve the accuracy of axle weight measurements between calibrations. For example, successive sample means could be calculated in turn, or a long-term moving average of the samples could be derived. When there was a significant difference between the sample means and the population mean, the system could just record the fact and do nothing, or it could alert operating personnel that the equipment was in need of recalibration. Alternatively, the system could automatically readjust the calibration factor by a small increment in the required direction. These fluctuations in the calibration factor could go unrecorded, or, more likely, would be recorded so that the weight data could subsequently be "unadjusted" should the need arise.

The concept of self-calibration appears to offer the potential for improved long-term accuracy of WIM systems. However, before the technique can be fully implemented, further detailed analysis of weight and classification data will be necessary, together with practical testing in the field to prove the concept. Castle Rock Consultants is currently addressing this need under contract to the states of Iowa and Minnesota.

ACCURACY EVALUATION

The accuracy of WIM systems can be directly determined by obtaining values for the mean and standard deviations of the PD or AD distributions, for both individual axle and gross vehicle weights, at various points in time following the initial system calibration. As with the initial calibration, it is impossible to measure the standard deviations directly, because not every vehicle that crosses the site can be checked. Consequently, a sample must again be taken, and the standard deviation of the sample used to obtain an unbiased estimate of the standard deviation of the population. It is possible to estimate confidence limits on this estimate of the population standard deviation. According to Spiegel *(9)*, the SE of this standard deviation is given by

$$\mathrm{SE} = \frac{\mathrm{SD}}{(2n)^{1/2}}$$

Therefore, for an initial calibration using 150 trucks (400 observations), 95 percent confidence limits on the standard deviation would be about ± 0.7 percent.

Each sample can be divided into weight ranges to detect differences in errors between weight ranges and statistical tests performed to identify significant differences between ranges.

Following the initial calibration, a second sample of weight measurements may be taken to observe whether any significant change in calibration or accuracy has occurred. A change in calibration will lead to different sample means, and changes in random scatter will lead to different sample variances. The two-tailed Students t-test can be used to determine, at any desired level of confidence, whether the means of two samples drawn from the same population are significantly different.

In the t-test, the null hypothesis is that the two means are equal and any actual numerical difference is due only to that amount of scatter expected when sampling a normal distribution. The t value is defined by

$$t = \frac{X_1 - X_2}{S_d}$$

where X_1 = sample mean of the calibration sample and X_2 = sample mean of the second sample. S_d is defined by

$$S_d = \left[\frac{(N_1 - 1)\,S_1^2 + (N_2 - 1)\,S_2^2}{N_1 + N_2 - 2}\right]^{1/2} \left[\frac{1}{N_1} + \frac{1}{N_2}\right]^{1/2}$$

where N_1, N_2 are the sample sizes, and S_1^2, S_2^2 are the sample variances.

The sample size N_2 can be estimated for particular significant differences in the means and for particular confidence levels. If a difference greater than ±2 percent is required at 95 percent confidence levels, for example, the sample size N_2 needed can be estimated as follows:

Assume N_1 (original calibration) = 400 axles and that the standard deviations of the samples S_1, S_2 are both 10 percent, then

$$S_d = 10\% \left(\frac{1}{400} + \frac{1}{N_2}\right)^{1/2}$$

If detecting differences in the means at a 0.05 level of significance is desired, then statistical tables show that the critical values of t are ± 1.96 in this two-tailed test. That is, if $t > 1.96$ or $t < -1.96$, the difference in the means was not due to the random scatter expected when sampling a normal distribution but was because the population of the two samples were different. In this case it would mean that the calibration had changed, leading to a systematic bias.

Using these values to find the minimum number for N_2 gives

$$t = \frac{2\%}{S_d} = 1.96, \text{ or } S_d = \frac{2\%}{1.96}$$

As it has already been estimated that $S_d = 10\% \,[(1/400) + (1/N_2)]^{1/2}$, then solving for N_2 yields N_2 = 126 axles. This

number of axles corresponds to about 50 trucks. Therefore, a second sample of 50 random trucks should be sufficient to detect a 2 percent change in the calibration, with 95 percent confidence.

PERFORMANCE STANDARDS

With the variety of WIM systems currently available, different accuracy levels can be obtained. Low-speed systems are capable of high-accuracy levels, whereas high-speed systems are relatively less accurate. As indicated earlier, users frequently want to infer static weights from dynamic weights so that the difference between the two measured weights is often treated as an integral part of the WIM system error. Despite the influence of highway and vehicle characteristics, it is generally recognized that there should be WIM performance specifications within which the WIM systems should operate.

This issue is currently being addressed by a research contract being undertaken as part of the Heavy-Vehicle Electronic License Plate (HELP) program, for which Castle Rock Consultants is a management consultant. The Development of Weigh-in-Motion Performance Specification contract involves the testing of different WIM systems in the laboratory and field, and the definition of performance criteria including accuracy, durability, and reliability. These criteria will be defined as recommended levels of performance and will be a quantitative assessment of the characteristics of WIM systems that could be used in the HELP program.

A possible approach for accuracy specification has been proposed by the authors. The approach would involve the specification of different accuracy limits according to the axle weight. At lower axle weights, below 10,000 lb for example, the accuracy is probably best specified as an absolute value, whereas beyond this threshold a percentage is more appropriate. This would lead a funnel within which the measurements should lie. An absolute tolerance of 1,000 lb and a percentage tolerance of 10 percent above 100,000 lb might be appropriate values (Figure 2). The percentage of measurements falling within the funnel would provide an indication of the system's ability to meet the accuracy performance specification.

Although this approach does not eliminate the influence of highway, vehicle, and environmental factors, it does appear to offer a suitable measure of operating performance. Further work would be necessary to establish acceptable values for the funnel dimensions.

CONCLUSIONS

Outlined in this paper are current WIM calibration techniques and the approach of self-calibration is developed as an aid to improved accuracies between regular calibration exercises. Analysis of TWS data has demonstrated the potential of the self-calibration concept, which relies on the fact that loads on certain axles of particular truck classes show relatively little variation, regardless of loading condition. Currently, work is in progress to implement self-calibration of WIM in Iowa and Minnesota.

The statistical design of calibration and accuracy evaluations has also been investigated. In particular, techniques for determining the sample sizes of measurements required to achieve specified accuracy and confidence levels have been outlined. Using such techniques, statistically valid evaluations can be readily performed to assess the calibration and accuracy of WIM systems.

An approach to the definition of a performance specification, or standard, has been proposed as part of this paper. At low axle weights, an absolute accuracy tolerance would seem to be appropriate, with a percentage tolerance for higher axle weights. This approach, producing a funnel-shaped tolerance band, may allow WIM systems to be specified and compared in similar ways.

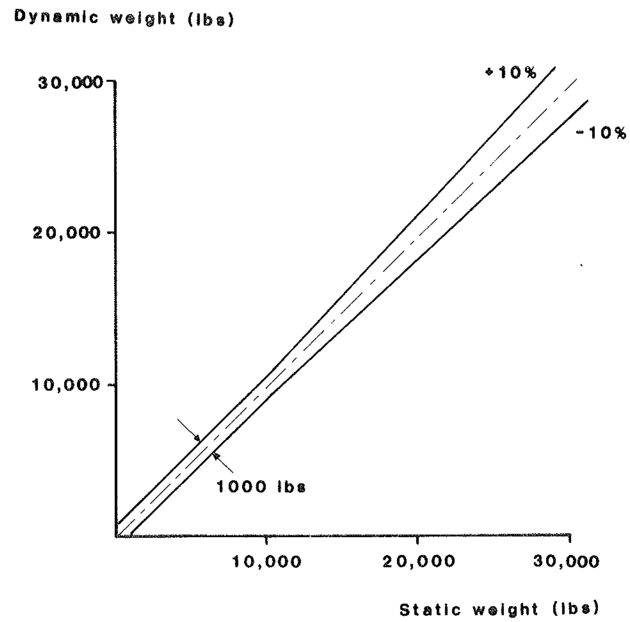

FIGURE 2 WIM performance specification.

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Accuracy and Tolerances of Weigh-in-Motion Systems

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A systematic study of in-motion weighing of some 800 trucks that were selected from the traffic stream on I-10 near Seguin, Texas, yielded data sets that were analyzed to define the attainable accuracy within which wheel, axle, axle-group, and gross-vehicle weights could be estimated by a properly calibrated in-pavement weigh-in-motion system. Each truck that was weighed passed successively over the Radian weigh-in-motion system transducers at high (±50 mph), intermediate (±30 mph), and low (≤10 mph) speed and then stopped on a special axle/wheel reference scale for successive static weighing of each wheel. Tolerances for a 95 percent confidence level were derived after the system had been calibrated to yield a zero mean of differences in the weigh-in-motion wheel weight estimates and the corresponding static wheel weights. The concept of use tolerances, which allow for the probable error in both the static weight measurement and the weigh-in-motion weight estimate, is presented. Tolerances for high-speed weigh-in-motion, intermediate-speed weigh-in motion, and low-speed weigh-in-motion scales at the experimental site are tabulated.

Although weigh-in-motion (WIM) systems have been operational for two decades, the accuracy with which static vehicle loads can be estimated at high, intermediate, and slow traffic speeds when compared with static scale measurements has not been systematically investigated or documented for mixed traffic. Previous studies *(1, 2)* have addressed the accuracy of the Texas WIM system by analyzing data sets from test trucks. With the static weighing technique *(3)*, the overall accuracy of a WIM system is determined not only by the accuracy with which force measurements can be made by the system, but also by the signal-processing technique and by how the system is used. The systematic bias in weight estimates made by a WIM system can be reduced significantly, and the variability can be affected somewhat, if the system is properly calibrated at each site where it is used *(4)*.

The purpose of this paper is to discuss the observed accuracy and tolerances in weight estimates associated with a Radian WIM system when about 800 trucks were weighed by the system and on special static axle/wheel scales at three different speeds in a series of field experiments in Texas. The information that is presented is a valuable resource for consideration when selecting suitable equipment for various purposes and when defining appropriate tolerances for truck-weighing operations that are conducted either for collecting statistical data or for enforcement. The concept of use tolerances is discussed, and appropriate tolerance limits for WIM scales are suggested. These values are intended to incorporate all probable errors associated with using a particular weighing device and technique so that the selected device can be used with confidence.

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This paper is organized in the following order: first, a brief description of the experimenting program and the analyses of data are given. Then, the basic concept of use tolerances is presented along with some recommended tolerance limits for the WIM system used in this study.

EXPERIMENTAL PROGRAM

The weigh station adjacent to the eastbound lanes of I-10 at Milepost 616 east of Seguin, Texas, was selected as the experimental site for data collection. High-speed weigh-in-motion (HSWIM) scales were installed in the right-hand main lanes about 500 ft in advance of the exit-ramp gore of the weigh station. Speed over these scales averaged about 50 mph in the experiment.

Intermediate-speed weigh-in-motion (ISWIM) scales were placed in the straight section of the exit ramp 470 ft in advance of the low-speed weigh-in-motion (LSWIM) scales. The average speed over the ISWIM scales was observed to be 30 mph, and the rollover speed on the LSWIM scales was less than about 10 mph. The reference (axle/wheel) scales were placed 80 ft beyond the LSWIM scales on a straight level (longitudinal) section of the weigh station. All the WIM scales were supported by a Radian instrument system that was housed in a mobile laboratory trailer located opposite the ISWIM scales.

Profile of the Road Surface

Gross-vehicle weight and axle-group weights can be determined in several ways. The most accurate way requires the use of a multiple-section vehicle scale using single-draft weighing, whereby all wheels on the vehicle are weighed simultaneously while the vehicle is in static equilibrium. Because of the expense involved, such a vehicle scale was not made available to determine the gross-vehicle and axle-group weights of the trucks in this study. Another way to determine gross-vehicle weight and axle-group weights is to successively weigh wheels, axles, or axle groups on axle-load scales or wheel-load weighers with all the vehicle components motionless and in exactly the same relative position to one another at the time of each weighing. Theoretically, this condition of exact positioning can best be achieved on a perfectly smooth and horizontal surface that is free of any unevenness. In reality, however, a road surface of this type is almost impossible to construct and maintain because of economic factors. Displacement of any vehicle component between or during successive weighings

due to torque, braking, load-shifting, and the associated frictional forces, also causes redistribution of the gross-vehicle weight among the axles and wheels and results in inaccuracy in the gross-vehicle weight and the axle-group weights calculated by summing the successive measurements.

The existing straight, zero-grade section of the weigh station chosen for use in this study had a 3 percent cross slope to the left-hand side in the weighing lane. At the time the site was selected, the permanent axle-load scale had been installed in a shallow concrete pit with zero cross slope in the immediate vicinity of the scales. The asphalt concrete surface had been warped from the 3 percent cross slope before and beyond the shallow pit to transition to the level plane of the scale surface. This warped cross section was not shown on the plans and was not evident until construction of the reference scale pit was begun. Limited funds and time available for the study made it necessary to install the reference axle and wheel (AX/WHL) scale also at zero cross slope and to warp the adjacent surface into the 10-ft-long concrete approach aprons that were constructed before and beyond the scales. Figure 1 shows the longitudinal profile in each wheelpath at the site at the time when data collection began. The longitudinal profile at the center of the vehicle path was excellent, but the warping of the cross slope at the scale pits was a matter of concern as it could possibly have affected wheel weights adversely. The effect of the local warping of cross slope was not expected to be as pronounced on axle, axle-group, and gross-vehicle weights, however.

After the first 2 days of data taking, premixed asphalt concrete was used to replace the existing asphalt concrete surface and a weighing lane with zero cross slope before, between, and beyond the reference and the permanent scales was built. This level surface held up well under truck traffic for 2 days of data taking, but rutted considerably in the hot summer weather by the fifth day of data taking.

Later in June 1984, the premixed surface material was removed and replaced with hot-mixed, hot-laid asphalt concrete to form a level lane (longitudinally and transversely) approximately 400 ft long. The LSWIM scales were removed before the leveling and reinstalled afterwards. An additional 100 trucks were weighed on the AX/WHL and LSWIM scales on July 6, 1984, after leveling the surface to within about 0.02 ft for 380 ft surrounding these three scales.

Description and Operational Features of Equipment

The nomenclature and operating features of each scale are given below in the order in which each truck passed over them. The weigh-in-motion system was supported by a four-lane Radian system that was developed especially for the study and for subsequent use in Texas for data collection.

High-Speed Weigh-in-Motion (HSWIM)

This scale used two flush-mounted wheel-force transducers, each 53 × 18 in. in plan dimensions, centered transversely in each wheelpath so that the tires traveled along the 18-in. dimension. Each transducer was supplied with ±1 percent maximum tolerances in electrical output signal. The analog signal was digitized and processed by a microcomputer in real time on site to convert the measured dynamic wheel force to an estimate of static wheel weight. Speed and axle-spacing computations were also made by the WIM system from inductance loop-type vehicle-presence detector signals. Thus, as a truck passed over the WIM scales, time of day, speed, axle spacing, wheelbase, wheel weights, axle weights, axle-group weights, gross-vehicle weights, bridge-formula compliance, and vehicle class were determined automatically, displayed on the video screen, and recorded on magnetic disk in digital format. Instruments for the WIM system were housed in a mobile laboratory trailer.

Intermediate-Speed, Weigh-in-Motion (ISWIM)

This scale was the same as HSWIM, but it was used at a slower speed (approximately 30 mph).

Low-Speed, Weigh-in-Motion (LSWIM)

This scale also was the same as HSWIM but each truck rolled over it at a speed of less than about 10 mph. Furthermore, on

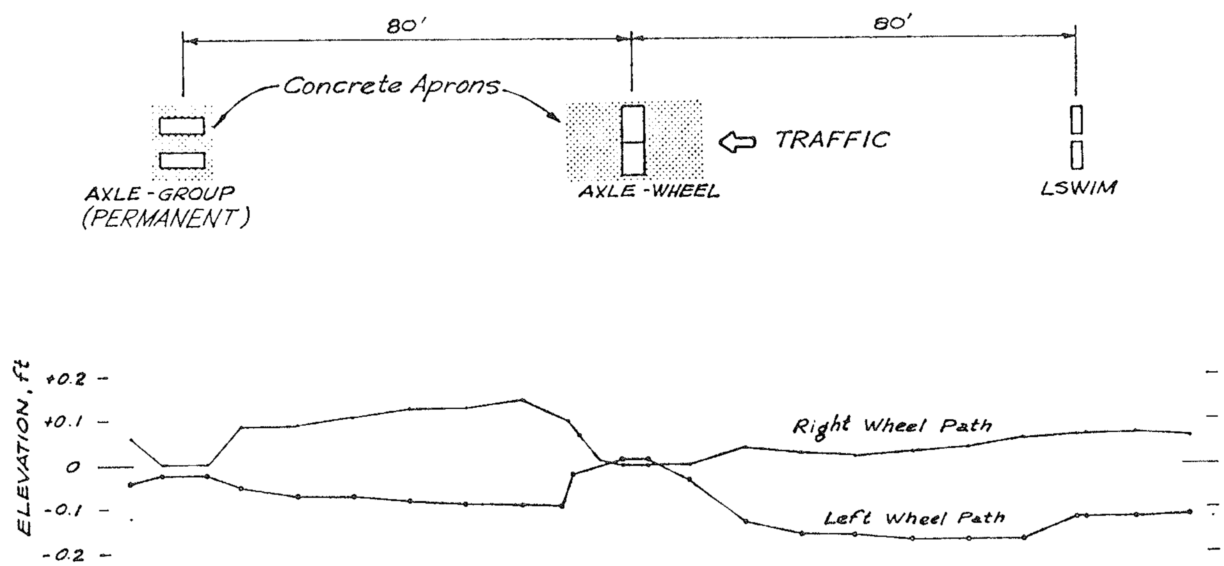

FIGURE 1 Longitudinal profile in each wheelpath of the weighing lane at beginning of tests.

the last day of data taking (July 6), this scale system was calibrated in place with ten 1,000-lb test blocks furnished by the Texas Department of Agriculture, Weights and Measures Section. The LSWIM scales performed within ±1 percent overall system tolerances under dead-weight loading.

Reference Axle and Wheel Scale (AX/WHL)

This scale consisted of two scale platforms, each 4 × 6 ft in plan dimensions, arranged side-by-side and mounted flush with the road surface so that wheels rolled along the 4-ft dimension; thus, each wheel on an axle could be weighed separately when the axle was positioned on the pair of scales. The design of the scale uses all flexure types of devices to transfer forces to the levers and finally to a single strain-gauge load cell. The load-receiving surface is supported by a tabular metal frame that deflects very little under load. The manufacturer states that one part in 5,000 (0.02 percent) tolerances are attainable with the scale. Under dead-weight testing using a series of 1,000-lb test blocks, the scale always indicated correctly within the 20-lb increment that was selected for use in the study. Time of day, wheel weights, axle weights, axle-group weights, and gross-vehicle weights from these scales were printed on a hard-copy tape by a microcomputer.

Traffic Control and Data Collection

Traffic through the weigh station was controlled by uniformed officers of the Department of Public Safety (DPS). One DPS officer and one State Department of Highways and Public Transportation (SDHPT) person were stationed approximately 2 miles upstream of the weigh station. Selected trucks were directed to stop on the shoulder by the officer; all other traffic was allowed to continue on the main lanes. A serialized identification number was attached to the front windshield of each selected truck by the SDHPT official. The trooper instructed each driver how to proceed through the weigh station and released a truck only when it could be processed at the weigh station without having to stop before crossing the LSWIM scale. The release time was coordinated via radio contact with the weigh station.

When released by the trooper, each truck traveled in the right-hand lane of I-10, passed over the HSWIM scale at about 55 mph, exited, and passed over the ISWIM scale at approximately 30 mph. Each truck was then stopped approximately 20 ft in advance of the LSWIM scale and the driver was instructed to roll slowly over the LSWIM scale and stop with the front axle on the AX/WHL scale. Another trooper instructed the driver to release the brakes after stopping each axle on the AX/WHL scale and wait for weighing. A weight reading was taken only after no appreciable change in the indicated weight was observed.

ANALYSIS OF WIM DATA

The sum of the vertical forces exerted on a perfectly smooth and level road surface by the perfectly round and dynamically balanced rolling wheels of a vehicle (i.e., an ideal vehicle) at a constant speed in a vacuum is exactly equal to the gross weight of the vehicle. In reality, these ideal conditions do not exist. However, if the deviations from the ideal are small, static weight estimates of acceptable precision and accuracy for certain purposes can be obtained from samples of dynamic wheel force. The field data collected in the experimental program are representative of actual truck traffic conditions under normal road and environmental conditions. The data sets are analyzed to determine mainly the accuracy with which static wheel, axle, axle-group, and gross-vehicle weights can be estimated from dynamic wheel forces measured with a properly calibrated WIM system at three different speeds. Axle weight and axle-group weight have been taken as the sum of all wheel weights for the particular axle or axle group under consideration, and gross-vehicle weight has been computed as the sum of all axle and axle-group weights on a truck or truck-trailer combination.

Graphical and statistical methods, including regression techniques, are used here for the comparison and correlation analysis of the data sets. Static weights that are used as a basis for comparison were obtained from the AX/WHL scale. This scale was accurate under dead-weight testing and weighed a test truck that made more than 60 runs over the scales very consistently throughout the 6 days of data-taking sessions. Because the number of trucks weighed was large and the mix of truck types in the sample was similar to the mix in the total traffic stream, the sample can be considered representative of the population of trucks that would be weighed in practice.

Three different data sets, one each taken on June 6 and 11, 1984, over all three WIM scales, and a third set taken on July 6, 1984, only over the LSWIM scale, are analyzed and presented in the following sections.

Graphical Representation

In the graphical approach, the weight data from the static weighings are plotted on the horizontal axis, labeled AXLE/WHEEL SCALE, and the corresponding weight for each vehicle as estimated by the WIM system at each speed is plotted along the vertical axis (labeled WIM SCALE) in each figure. Bounds of +10 percent and −10 percent difference in the WIM-estimated weight and that obtained from the static AX/WHL scale are shown as divergent sloping lines in each figure. Dot-dash lines on these figures indicate the legal weight limits. In another graphical approach the relative difference in the WIM-estimated weight, which is calculated and expressed as a percentage of the weight measured by the reference scale, is plotted against that of the corresponding reference weight.

Statistical Procedures

Statistical tests of normality *(5, 6)* indicate that the frequency of relative differences in WIM-estimated weights can be considered to be normally distributed; therefore, by applying the properties of a normal frequency distribution, certain inferences can be drawn from analysis of the data sets. The sampled data are considered to be representative samples drawn from a large parent population.

For further analysis of the data, in order to examine the relationship between the WIM estimates of the static weights and the respective weights from the AX/WHL scale numerically, a linear regression analysis is used. For each data set, the regression is performed on the WIM-estimated weights against

the corresponding observed weights from the static scale. Although the obvious purpose of this analysis is to determine the accuracy and precision, on the average, associated or attainable with WIM systems for predicting the true weights from samples of dynamic wheel forces, the equations are derived by using weights measured from the AX/WHL (reference) scale to predict weights from the WIM scales. This is necessary because, in a normal regression equation $y = b_0 + b_1x$, the predictor or independent variable x is assumed to be virtually error free, whereas the response or dependent variable y is not. Thus, weight determined from the reference scale is taken as the predictor variable x in developing the needed regression equation. The fitted straight line, in essence, provides a calibration curve for the WIM scales, related to the static weight data from the reference scale. The problem of estimating true weight from a WIM system measurement of dynamic force is called in statistics the inverse regression problem and is fully documented in studies by Halperin *(7)* and Ostle and Mensing *(8)*. So the equation for a given y, namely y_0, may be inverted, or solved for the inverse estimate of x, by solving the following equation for x_0

$$y_0 = b_0 + b_1x_0,$$

namely

$$x_0 = (y_0 - b_0)/b_1$$

so that force measurements from the WIM scales can be used to estimate the static weight that would be expected to result from weighing on the reference scale.

Results of the regression analysis are tabulated for axle-group and gross-vehicle weights in the following paragraphs. These regression equations were developed for each WIM scale—LSWIM, ISWIM, and HSWIM—used in the experiment. For cases in which it is known or in which it has been found empirically that the standard deviation of the untransformed response y, σ_y, say, is a function of the mean value, $\mu = E(y)$, a natural-log transformation of the data is used in the analysis. The coefficient of variation (c.v.), which is a measure of the precision with which true weight can be estimated by the equation, is computed for each equation. As previously explained, the coefficients are computed on the basis of the reference scale weight being the predictor variable; therefore, small inaccuracies can result from applying the coefficients to the inverted equations. These inaccuracies, however, cannot possibly be large because of the relatively small scatter in the untransformed or transformed weight information. The c.v.'s can be treated as standard deviations of the relative difference in weights. That is, true weights estimated by the regression equations from weight measurements by the WIM scales will yield estimates within ($\pm 2 \times$ the coefficient of variation) of the actual weight values approximately 95 percent of the time (i.e., within the 95 percent confidence limits). The regression coefficient or the slope of the line, on the other hand, is the measure of correlation or agreement between the WIM estimates of the static weights and the corresponding measurements from the AX/WHL scale. A slope of 1.0 and a c.v. equal to zero percent would result if perfect agreement existed between the two sets of weight readings.

Gross-Vehicle Weights

Figures 2, 3, and 4 illustrate the variability that was observed in gross-vehicle weight estimates for 61 trucks when each truck was weighed at three different speeds (low $\leq$ 10 mph, intermediate = approximately 30 mph, and high = approximately 50 mph) on June 11, 1984, by three properly calibrated WIM scales. Each graph illustrates the relationship between the WIM

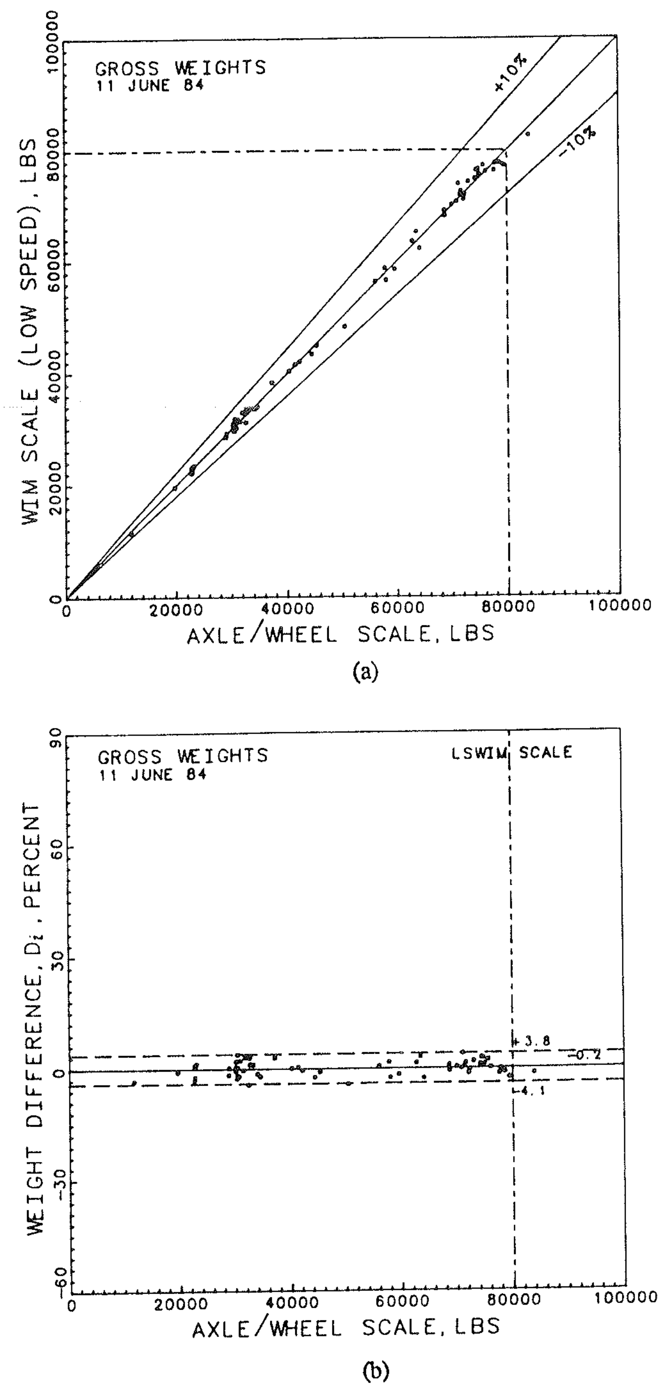

FIGURE 2 Gross-vehicle weight estimates for 61 trucks crossing the LSWIM scale *(a)* at less than 10 mph versus weights summed from the AX/WHL scale and *(b)* percent difference in gross-vehicle weight estimates from the LSWIM scale with reference to the static weights.

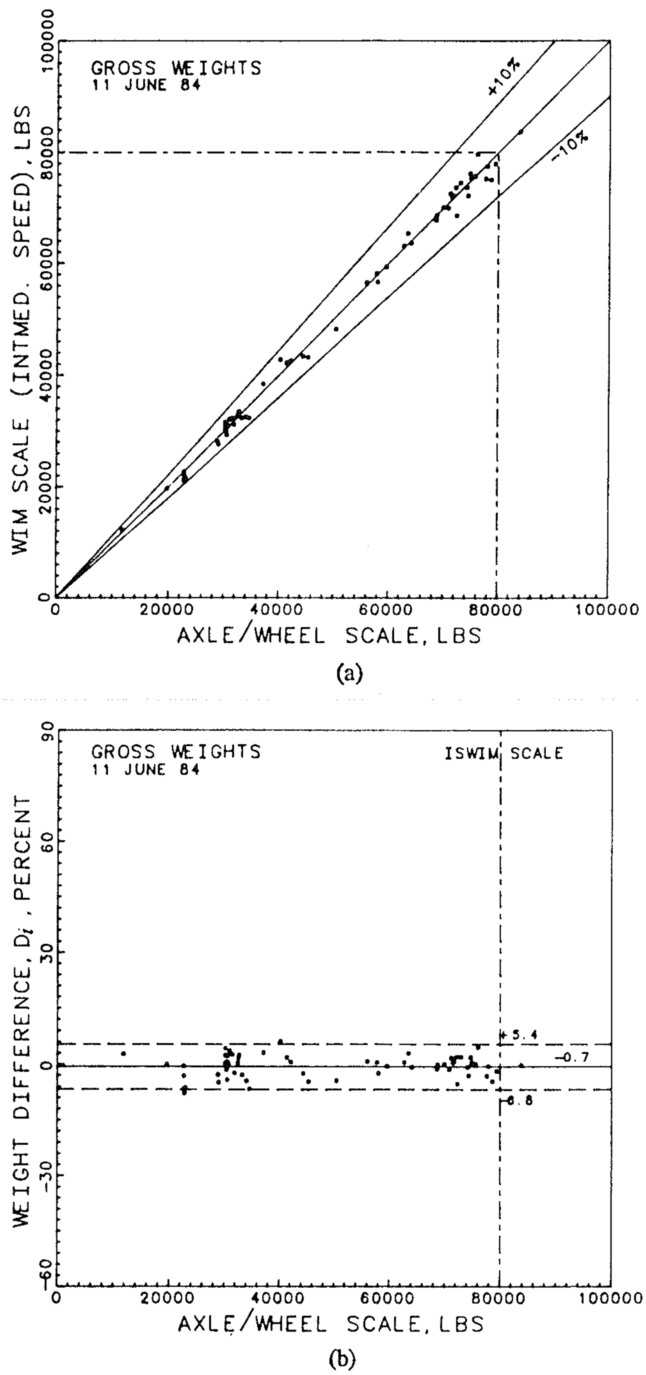

FIGURE 3 Gross-vehicle estimates for 61 trucks crossing the ISWIM scale *(a)* at about 30 mph versus weights summed from the AX/WHL scale and *(b)* percent difference in gross-vehicle weight estimates from the ISWIM scale with reference to the static weights.

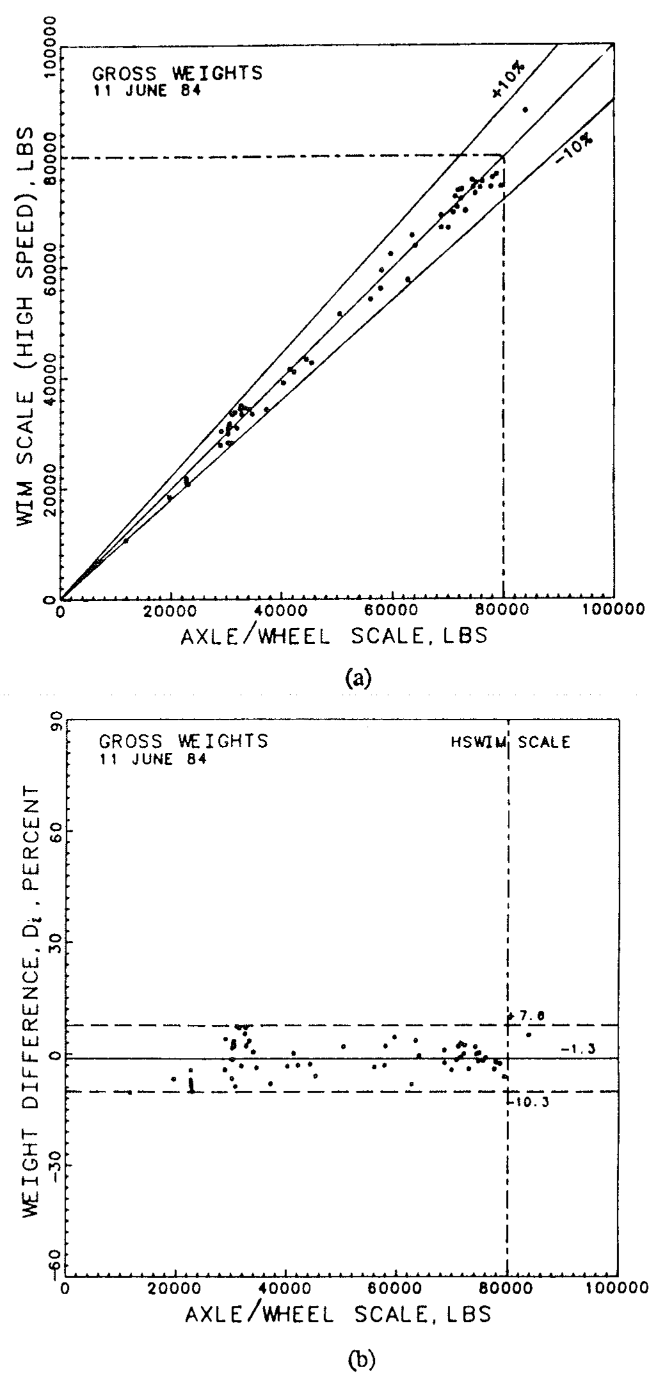

FIGURE 4 Gross-vehicle weight estimates for 61 trucks crossing the HSWIM scale *(a)* at about 55 mph versus weights summed from the AX/WHL scale and *(b)* percent difference in gross-vehicle weight estimates from the HSWIM scale with reference to the static weights.

system weight estimates and the corresponding weights from the AX/WHL reference scale. The static gross-vehicle weight that was used for reference was taken as the sum of the weights of all axles on the vehicle after each axle was weighed in sequence on the static AX/WHL scale. Careful examination of each of the data sets was made to check for abnormalities in weight data and a few (fewer than five) extreme outlying points were removed with discretion from the data sets. Implications such as a tire partially or fully off the WIM scale guided this process.

As shown in these figures, if there were perfect agreement between the two weights, all the plotted points would lie exactly on the 45° line of equality. The pattern of data points shown in these three figures indicates that there was a small but consistent increase in the range of gross-vehicle weight difference as the speed of the vehicles being weighed by the WIM system increased. For all three scales, the data points are clustered rather evenly with small scatter about the 45° line of perfect agreement. The gross-vehicle weights from the

TABLE 1 SUMMARY STATISTICS OF WIM GROSS-VEHICLE WEIGHTS AS COMPARED AND CORRELATED WITH THE AX/WHL SCALE WEIGHTS FOR SPEEDS AND TIMES SHOWN

DATE	SPEED AT WIM SCALES	STATISTIC				
		MEAN WEIGHT, LBS	MEAN OF DIFFERENCES (MEAN OF ABSOLUTE DIFFERENCES), %	95% CONFIDENCE RANGE, $\hat{\mu} \pm 2\hat{\sigma}$, %	REGRESSION ANALYSIS	
					SLOPE	C.V.*
June 11, 1984 n = 61	LSWIM (10 mph)	49570 (49600)+	-0.2 (1.6)	-4.1 to +3.8	1.00003	2.0
	ISWIM (30 mph)	49310	-0.7 (2.4)	-6.8 to +5.4	0.99494	2.8
	HSWIM (55 mph)	49080	-1.3 (3.8)	-10.3 to +7.6	0.99054	3.8
June 6, 1984 n = 60	LSWIM (10 mph)	38870 (39200)+	-1.3 (2.8)	-7.8 to +5.2	0.99344	2.6
	ISWIM (30 mph)	39000	-0.8 (2.8)	-7.7 to +6.1	0.99729	3.2
	HSWIM (55 mph)	38640	-2.0 (3.8)	-10.9 to +7.0	0.98803	4.0
July 6, 1984 n = 86	LSWIM (10 mph)	43900 (44180)+	-0.6 (2.6)	-6.8 to +5.7	0.99174	2.8

* Coefficient of Variation, %
\+ Reference Scale Mean Weight

HSWIM scales are, on the average, 1.3 percent lower than the respective static weights. Several light trucks produced large negative weight differences in terms of percentage; these had a rather large influence on this mean value. Although the dynamic effects of vehicle/road/WIM-system interaction on these gross-vehicle weights tend to be greater at higher speeds, virtually all the WIM estimates of gross-vehicle weights at high speed differed less than 10 percent from the observed static gross-vehicle weights (see Figure 4).

Results of the regression analysis, along with the statistical inferences drawn from the sample distribution of the relative difference in gross-vehicle weights, are summarized in Table 1. A linear regression equation (with zero intercept) was developed for each of the three WIM scales used in the experiment. The regression coefficient (i.e., slope of the line) and c.v. are also presented in this table. The slope and the coefficient of variation for each regression equation are measures of the accuracy with which estimates of static gross-vehicle weight can be predicted by the equation. It can be concluded, for example, that approximately 95 percent of the weight observations would produce estimates of static weight from HSWIM scales that would be within [±2 (c.v.) = ±2 (3.8 percent) = ±7.6 percent] of the actual values of the static gross-vehicle weights. The respective accuracies for the LSWIM and ISWIM scales are ±4 and ±5.8 percent. Or, without using a regression equation, gross-vehicle weights can be predicted with 95 percent confidence within ±4.0, ±6.0, and ±9.0 percent for trucks running over the scales at speeds of 10, 30, and 55 mph, respectively [see the confidence bands in Figures 2*(b)*, 3*(b)*, and 4*(b)*, respectively].

The value of the slope of the regression line, on the other hand, is a good indication of how well the static gross-vehicle weights are predicted by the estimated weights from the sampled dynamic wheel forces by the WIM scales. For the HSWIM scale, for example, the value of the slope of the regression line is 0.99054. This figure is very close to 1.0 and it implies that, on the average, the system makes accurate predictions of gross-vehicle weights. The respective values for LSWIM and ISWIM, respectively, are 1.00003 and 0.99494. Again these numbers are very close to 1.0, indicating that a small improvement in predictive accuracy can, on the average, be achieved by applying the regression technique. The confidence bands are reduced slightly.

The observed differences in the WIM-estimated gross-vehicle weights and the comparable static weights cannot be attributed entirely to WIM system error or to inaccuracy in the WIM system. Part of the difference comes from the redistribution mechanism of the gross-vehicle weight among the axles on the vehicle as it moves into different positions and stops for successive weighing of each axle on the static reference scale. This redistribution, which is governed to a large extent by the interaction of the vehicle with the road surface, the scale, and the atmosphere, occurs continually as the vehicle moves over the WIM system scales. Additionally, the dynamic behavior of the various interconnected vehicle components contributes to the magnitude of this difference at the time of weighing. The static gross-vehicle weights calculated from the reference (AX/WHL) scale are not without error because of the method of successive weighing that was used.

Axle-Group Weights

The total weight on a group of closely spaced axles is important in the engineering design of pavement and bridge structures and also in enforcement weighing. The WIM and AX/WHL scales indicated the weight of each wheel. Axle-group weights were calculated from these scales by summing the weights of all wheels on the axles in the group.

The calculated values for all axle-group weights, when each axle was weighed on LSWIM, ISWIM, and HSWIM scales, indicated that there was a small but consistent increase in the range of axle-group weight differences as the speed of the vehicles being weighed by the WIM scales increased. Statistical tests indicate that the relative difference in axle-group weights computed from the WIM estimates with reference to those from the AX/WHL scale, are normally distributed in a

TABLE 2 SUMMARY STATISTICS OF WIM AXLE-GROUP WEIGHTS COMPARED AND CORRELATED WITH AX/WHL SCALE WEIGHTS FOR SPEEDS AND TIMES SHOWN

DATE	SPEED AT WIM SCALES	STATISTIC				
		MEAN WEIGHT, LBS	MEAN OF DIFFERENCES (MEAN OF ABSOLUTE DIFFERENCES), %	95% CONFIDENCE RANGE, $\hat{\mu} \pm 2\hat{\sigma}$ %	REGRESSION ANALYSIS	
					SLOPE	C.V.*
June 11, 1984 n = 178	LSWIM (10 mph)	16990 (17000)+	-1.0 (3.7)	-10.0 to +8.0	1.00594	4.0
	ISWIM (30 mph)	16900	-0.7 (3.8)	-10.6 to +9.2	0.99538	4.4
	HSWIM (55 mph)	16820	-1.1 (5.6)	-15.7 to +13.4	0.99052	6.7
June 6, 1984 n = 171	LSWIM (10 mph)	13640 (13750)+	-1.9 (4.8)	-13.4 to +9.7	0.99962	5.0
	ISWIM (30 mph)	13690	-0.8 (4.6)	-12.6 to +11.0	0.99888	5.4
	HSWIM (55 mph)	13560	-1.6 (6.1)	-17.7 to +14.6	0.98754	6.7
July 6, 1984 n = 242	LSWIM (10 mph)	15600 (15700)+	-0.8 (3.9)	-11.4 to +9.8	0.99934	4.3

* Coefficient of Variation, %
\+ Reference Scale Mean Weight

TABLE 3 SUMMARY STATISTICS OF WIM AXLE WEIGHTS COMPARED AND CORRELATED WITH THE AX/WHL SCALE WEIGHTS FOR SPEEDS AND TIMES SHOWN

DATE	SPEED AT WIM SCALES	STATISTIC		
		MEAN WEIGHT, LBS	MEAN OF DIFFERENCES (MEAN OF ABSOLUTE DIFFERENCES), %	95% CONFIDENCE RANGE, $\hat{\mu} \pm 2\hat{\sigma}$, %
June 11, 1984 n = 280	LSWIM (10 mph)	10800 (10800)+	-0.1 (4.6)	-11.8 to +11.7
	ISWIM (30 mph)	10740	-0.1 (5.5)	-14.7 to +14.6
	HSWIM (55 mph)	10690	-0.1 (6.6)	-17.8 to +17.7
June 6, 1984 n = 253	LSWIM (10 mph)	9220 (9300)+	-0.6 (5.3)	-13.9 to +12.7
	ISWIM (30 mph)	9250	-0.5 (5.5)	-14.6 to +13.7
	HSWIM (55 mph)	9170	-0.5 (7.4)	-19.8 to +18.8
July 6, 1984 n = 367	LSWIM (10 mph)	10290 (10350)+	-0.1 (4.7)	-13.1 to +13.0

\+ Reference Scale Mean Weight

statistical sense. Therefore, some important statistical inferences were developed from analysis of the three data sets mentioned previously; these are tabulated in Table 2. These statistics can be interpreted to indicate that accuracies of about ±9, ±10, and ±14 percent can be expected when comparing LSWIM, ISWIM, and HSWIM estimates of axle-group weights with the corresponding weights from the static reference scale, respectively, at 95 percent confidence level. Or, using the regression equation estimates just described, axle-group weights can be predicted at the same level of confidence within ±8.0, ±8.8, and ±13.4 percent.

Axle and Wheel Weights

Summary statistics for axle and wheel weights are given in Tables 3 and 4, respectively. These results further support the fact that the distribution of weight among the axles of a vehicle changes as the vehicle moves over the road surface and stops for successive weighing of axles and wheels on static scales.

TOLERANCES

Concept

In dealing with weight measurements, a distinction should be made between accuracy and precision. Accuracy is the degree of conformity of a measurement to a standard or to a true value. Precision, on the other hand, refers to the exactness with which a measurement is made. A measurement can be precise without necessarily being accurate. Errors in precision are generally random or accidental and can therefore be explained by applying appropriate statistical concepts and techniques. Errors in

TABLE 4 SUMMARY STATISTICS OF WIM WHEEL WEIGHTS COMPARED AND CORRELATED WITH THE AX/WHL SCALE WEIGHTS FOR SPEEDS AND TIMES SHOWN

DATE	SPEED AT WIM SCALES	STATISTIC		
		MEAN WEIGHT, LBS	MEAN OF DIFFERENCES (MEAN OF ABSOLUTE DIFFERENCES), %	95% CONFIDENCE RANGE, $\hat{\mu} \pm 2\hat{\sigma}$, %
June 11, 1984 n = 560	LSWIM (10 mph)	5400 (5400)+	0.0 (8.7)	-21.8 to +21.8
	ISWIM (30 mph)	5370	0.0 (6.8)	-17.8 to +17.8
	HSWIM (55 mph)	5350	0.0 (8.4)	-22.3 to +22.3
June 6, 1984 n = 506	LSWIM (10 mph)	4610 (4650)+	0.0 (8.8)	-22.6 to +22.6
	ISWIM (30 mph)	4630	0.0 (8.1)	-21.3 to +21.3
	HSWIM (55 mph)	4580	0.0 (10.5)	-27.2 to +27.2
July 6, 1984 n = 734	LSWIM (10 mph)	5140 (5180)+	0.0 (6.0)	-16.0 to +16.0

+ Reference Scale Mean Weight

accuracy are usually systematic and can frequently be minimized or eliminated by adjustment or calibration of a properly designed weighing device that has good precision. In using a weighing device that has systematic errors that cannot be eliminated by calibration, the systematic errors combine with the random errors to determine the overall accuracy with which weight can be measured by the device.

In recognition of the fact that errorless performance of mechanical or electromechanical equipment is unattainable, tolerances are established to define the range of inaccuracy within which such equipment will be allowed to perform and still be approved for official use in a jurisdiction. The U.S. Department of Commerce, National Bureau of Standards, has set out code requirements [*NBS Handbook 44* (1986)] for static scales (but not yet for WIM scales) in official use for the enforcement of traffic and highway laws or for the collection of statistical information by government agencies. Acceptance tolerances are defined in the code and are applied to new or newly reconditioned or adjusted equipment. Maintenance tolerances, which are generally twice the acceptance tolerances, are applied to the equipment that has been in service for some time; these tolerances define the maximum variation in accuracy that will be permitted when the equipment is tested against an official standard. The official standard for verifying the performance of static scales is a set of standard test weights of known value.

Use Tolerances

In-motion weighing involves two processes: *(a)* sampling a dynamic tire force, and *(b)* using the sampled force to estimate the corresponding portion of the gross-vehicle weight that this tire would carry if weighed statically. Neither of these processes, nor the corresponding measurement of static tire force, can be performed without error. Therefore, not only basic tolerances, which protect the interests of both the users of the information obtained by WIM systems and the manufacturer of the system, but also use tolerances are needed. Use tolerances account for both the inherent variability in the physical phenomenon being estimated (i.e., static wheel force) and the accuracy with which a WIM system can possibly and practically perform each of the two processes mentioned above. As with static scales, the overall accuracy of a WIM system is determined partly by the accuracy that is attainable by the system itself and partly by how the system is used *(3)*. A number of site-specific conditions such as road profile, cross slope near the WIM transducers, interaction of the transducer/roadway system under dynamic load, and vehicle factors affect the overall accuracy of an installed WIM system.

The importance of on-site calibration for WIM systems is discussed by Lee et al. *(4)*. However, the inherent variability in weight data due to factors such as torque in the vehicle drive train, dynamic behavior of the various interconnected vehicle components, friction, and other factors, cannot be completely accounted for, even by a properly calibrated system. Therefore, use tolerances that recognize such variability must be used when interpreting and applying WIM-estimated weights for enforcement or for statistical data-collection purposes.

In the regression analysis already mentioned, it is assumed that the reference weight x (i.e., the predictor variable) is not subject to random variation, but that the WIM estimate y (the response variable) is. The regression model $y = \beta_1 x + e$ is considered in the analysis because the nonzero intercept term is physically difficult to explain and justify. Because the actual observed value of y varies about the true mean value with the unknown variance σ^2, a predicted value of an individual observation, which is given by $y = b_1 x$, has greater variation than σ^2. This means that a prediction interval for the particular outcome of a weight reading from the WIM scale can be defined. A prediction interval is one that contains y with a desired level of confidence. A one-sided (upper band width) 95 percent prediction interval for y at a fixed value x can be constructed. The use tolerance for a given data set is then determined by subtracting

the value of x (the reference weight) from the predicted value of the weight plus its upper prediction interval. The results from the regression models are given in Table 5.

TABLE 5 USE TOLERANCES FOR AXLE-GROUP AND GROSS-VEHICLE WEIGHTS FOR THE WIM SCALES (95 PERCENT CONFIDENCE LEVEL)

SPEED	AXLE-GROUP WEIGHT TOLERANCE (LBS)	GROSS-VEHICLE WEIGHT TOLERANCE (LBS)
LSWIM (< 10 mph)	+1100 (-1350 to +1350)*	+1650 (-2050 to +1950)
ISWIM (< 35 mph)	+1100 (-1550 to +1350)	+2000 (-3050 to +2450)
HSWIM (< 55 mph)	+1700 (-2400 to +2050)	+2650 (-4300 to +3250)

* Two-Tailed 95% Confidence Limits to Show Upper and Lower Limits of Tolerances

To apply the use tolerances to estimated weights from a particular WIM device, the user may calculate a probable minimum weight by subtracting the applicable tolerance value (e.g., at 95 percent confidence) from the WIM-estimated weight. He can then be sure that there is only a 5 percent probability that the estimated weight would be less than that calculated if it were measured on the reference scale. For example, a tandem-axle group weight is estimated by an LSWIM scale at 35,500 lb. The probable minimum weight would be 35,500 − 1,100 = 34,400 lb (see Table 5). An enforcement officer using the LSWIM system could charge that the axle-group weight was in violation of the 34,000-lb legal limit and be sure that there was only 1 chance in 20 that it would weigh less than 34,400 lb if weighed on the accurate reference scale.

SUMMARY

Statistical analysis of the performance of the Texas WIM system at different speeds indicates that a properly calibrated system can produce the following results compared with the respective weights from the AX/WHL reference scale (see Table 6).

TABLE 6 TOLERANCE VALUES FOR A CALIBRATED WIM SYSTEM

Speed at WIM Scale	Statistical Inference	Gross-Vehicle Weight (% difference)	Axle-Group Weight (% difference)
LSWIM (10 mph)	Mean of differences	−0.2	−1.0
	range for 95%	+3.8 to −4.1	+7.9 to −10.0
ISWIM (30 mph)	Mean of differences	−0.7	−0.7
	range for 95%	+5.4 to −6.8	+9.2 to −10.6
HSWIM (55 mph)	Mean of differences	−1.3	−1.1
	range for 95%	+7.6 to −10.3	+13.4 to −15.7

These values imply that tolerances of about ±4 percent, ±6 percent, and ±9 percent would be appropriate when interpreting LSWIM, ISWIM, and HSWIM estimates of the gross-vehicle weight from the static reference scale, respectively, if the WIM-estimated weight is expected to be within the chosen tolerance value for 95 out of 100 vehicle weighings. Likewise, tolerances of about ±9 percent, ±10 percent, and ±14 percent should be applied to WIM-estimated axle-group weights for the same level of confidence.

The results of these analyses also indicate that the performance of this WIM system is adequate for use *(a)* in gathering weight data at high speeds for statistical information, *(b)* as a means of sorting overweight trucks in enforcement programs, and *(c)* in weighing trucks at low speeds for legal evidence of weight-law violation (compared with the performance of the static axle-load scales and wheel-load weighers that are being used at the present time in enforcement programs). It is also concluded that the use tolerances for the properly calibrated LSWIM and ISWIM systems are lower than the corresponding use tolerances for all the static weighing devices *(4)* utilized in the field study.

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On-Site Calibration of Weigh-in-Motion Systems

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The importance of on-site calibration for weigh-in-motion systems has been illustrated by comparing weigh-in-motion weight estimates, made after calibrating the system by three different calibration-loading patterns, against corresponding wheel weights measured on a special static reference scale. Various truck types selected from the traffic on I-10 near Seguin, Texas, were included in the analysis and high, intermediate, and low speeds of in-motion weighing were considered. A pronounced improvement in the accuracy with which weights were estimated by the high and intermediate systems was achieved when six loaded five-axle tractor-trailer trucks, chosen randomly from the traffic stream, were used as the basis for calibration compared with multiple runs of the same loaded two-axle, single-unit test truck. The variability in weigh-in-motion weight estimates was not affected appreciably by the type of moving-vehicle loading that was used as the basis for calibration. Static-weight loading is recommended for low speeds of weigh-in-motion calibration, and moving-vehicle loading is recommended for practicable on-site calibration of higher-speed weigh-in-motion systems. Suggestions are offered on the types of trucks and the minimum number of wheel loads that should be used as the basis for on-site calibration.

State-of-the-art technology in in-motion weighing makes it possible to use measurements of the dynamic tire forces that are applied to the road surface by moving vehicles to estimate the weights of vehicles within certain tolerances. While the currently attainable tolerances are not considered to be acceptable for commercial weighing, they are adequate for applications of weigh-in-motion (WIM) systems for collecting statistical data and for aiding enforcement. With a WIM system, it is practicable to weigh, classify, and measure the speed of every vehicle that passes in each lane of a multilane highway during any chosen time period. Thus, virtually a 100 percent sample of traffic data for statistical purposes can be obtained, and the information can be transmitted immediately in real time, or at some future time, to locations remote from the WIM site via conventional communications networks. At present, WIM systems are applied in enforcement primarily for identifying individual vehicles that are suspected of being in violation of weight or size laws and for locating sites where relatively large numbers of probable weight, speed, or size violations occur.

The magnitude of acceptable error, or tolerance, in WIM weight estimates continues to be a matter of concern. The smallest practically attainable tolerance is, of course, the objective. A number of different factors can cause a WIM-system estimate of wheel weight to vary from the true static weight of the wheel *(1)*. Some of these factors are associated with the WIM system itself and others are related to the roadway, vehicle, and environmental conditions under which in-motion weighing is performed. For a WIM system to perform within small tolerances, the instrument system must make accurate measurements of the vertical component of the dynamic (continually changing) force that is applied to a smooth, level road surface by the tires of a moving vehicle and use these measurements as the basis for calculating an estimate of the static wheel weights. Proper force transducers and the related signal processing equipment are obviously essential components of such a system, along with the required computing power for interpreting the complex dynamic signals. A requisite for using the system is to have the roadway surrounding the transducers as nearly smooth and level as practicable.

It is well known that road surface roughness in the vicinity of WIM scales has a pronounced effect on the dynamic tire forces that result from the vehicle/road interaction *(2)*. Every vehicle will interact with the roughness differently, and vehicle speed will affect the dynamic forces to different degrees. Prevailing environmental conditions such as wind and ice can also affect dynamic wheel forces at a specific time and place, and cause variability in WIM weight estimates. Therefore, even though a particular type of WIM system meets specified performance tolerances at one particular site, it might not perform within the same tolerances at another site. Some of the variability and much of the systematic bias in WIM-system weight estimates that are due to roadway and environmental conditions can be removed or reduced by calibrating the system after it is installed at the site where it will be used. However, fundamental deficiencies in the design or operation of the WIM system itself cannot generally be overcome by calibration.

In this paper, some general concepts and techniques of on-site calibration of WIM systems are presented. The relative effectiveness of two on-site calibration techniques is demonstrated by applying the techniques to rather extensive in-motion-weighing data sets that were obtained in a series of field experiments conducted as part of the Rural Technical Assistance Program (RTAP) WIM Demonstration Program in Texas during the summer of 1984 *(3)*. Recommendations are given for practical on-site calibration of low-speed (LSWIM), intermediate-speed (ISWIM), and high-speed (HSWIM) weigh-in-motion systems.

GENERAL CONCEPTS

The load cells that are used as WIM wheel-force transducers can be calibrated individually in the factory under static load, but the response of the transducer/roadway/tire-loading system

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under dynamic loads cannot be easily evaluated in the laboratory. There is a complex interaction among the various components of this physical system that is unique for every location and vehicle load that is applied to the transducer.

A properly damped wheel-force transducer and a supporting instrument system that is capable of measuring accurately the vertical component of dynamic tire loads in the actual roadway environment are the essential hardware elements of a weigh-in-motion system. A software system that converts these dynamic force measurements into an estimate of the proportion of the gross vehicle weight that the wheel would carry if it were weighed statically must complement this hardware element for an overall WIM system to function.

A number of site-specific conditions such as road-surface roughness, grade, cross-slope near the WIM transducers, behavior of the transducer/roadway combination under dynamic load, and the speed and composition of traffic at the site affect rather significantly the overall accuracy with which a system can estimate static wheel loads. Every vehicle will interact differently; therefore, an on-site WIM-system calibration procedure is necessary if the best possible static weight estimates are to be made for the population of various vehicle types that will cross the WIM system at the site.

The objective of calibration is to make the weights estimated by the WIM system agree as closely as possible with the corresponding weights that would be measured by static scales. It is important to recognize that the proportion of the gross vehicle weight carried by each wheel of a vehicle changes as the vehicle moves over the road surface and stops on the static scale for weighing; thus the wheel force applied to a static scale can vary according to the relative position of the interconnected vehicle components at the time of weighing *(3)*. Perfect agreement between WIM weight estimates and static weight measurements is not expected because the quantity that is being estimated can vary with time and the position of the vehicle components when it is measured on static scales. By calibration, an attempt is made to make the mean value of WIM weight estimates agree as closely as possible with the best estimate of static weight that can be obtained feasibly in practice.

LOADING TECHNIQUES FOR CALIBRATION

Two basic types of loading can be used for on-site calibration of WIM systems: *(a)* static-weight loading, or *(b)* moving-vehicle loading. In the first type of loading for calibration, a known weight is applied to the WIM force transducer either by standard test weights (or force-reaction system) or by the wheels of a standing test vehicle. Standard test blocks provide a much more reliable reference weight than the standing test vehicle as the proportion of the gross-vehicle weight carried by any given wheel of the test vehicle changes as it moves onto the transducers and stops for weighing. In practice, however, it is sometimes difficult or expensive to use standard test blocks as a basis for calibration loading. A loaded test vehicle is usually easier to obtain for this purpose, but considerable care must be exercised in weighing each wheel of the test vehicle statically as well as in positioning the wheels on the WIM transducers. The static-weight loading technique is not generally appropriate for calibrating higher-speed WIM systems because the dynamic behavior of the moving vehicle must be considered as the vehicle interacts with the roadway surface and with the WIM transducers.

The moving-vehicle loading technique is applicable for calibrating intermediate- and high-speed in-motion weighing (ISWIM and HSWIM) systems wherein the dynamic interaction of the vehicle with the WIM system is much more pronounced. In this technique, a single test vehicle with known static wheel weights can make multiple runs over the WIM system transducers at a representative speed of traffic at the weighing site to produce a data set that defines the differences in the WIM-system weight estimates and the known static weights. Or different types of test vehicles with known wheel weights can each make multiple runs over the transducers to obtain a better representation of the various patterns of vehicle/roadway/WIM-system interaction that occur at the site. Alternatively, a single pass of several different trucks, each with known wheel weights, over the WIM system can provide a data set for determining on-site calibration settings for the WIM instrument system.

COMPARISON OF CALIBRATION LOADING TECHNIQUES

The importance of on-site calibration and the relative effectiveness of various calibration loading techniques are illustrated by the data shown in Tables 1 through 3. In these tables, summary statistical inference values from the comparison of a large number of weight estimates made by a Radian WIM system are presented after calibrating the system by three different loading techniques with the respective weights determined by weighing each wheel of the same vehicles statically on a special (two 4- × 6-ft platforms, side by side) axle-load reference scale (the AX/WHL scale). Differences in individual weight values were computed and expressed as a percentage of the reference scale weights. The mean of these percent differences is given along with another statistical value, $\hat{\mu} \pm 2\hat{\sigma}$ which defines the 95 percent confidence intervals into which an individual weight difference would probably fall if it were determined in the same way and under the same conditions that the sampled weight differences were determined.

Calibration of the WIM system for this comparative analysis involved the calculation and application of a single calibration factor (CF) that could be applied as a multiplier to the force signals from each WIM system wheel-load transducer to make the mean of the weight differences for all wheels weighed on each transducer equal zero with respect to the corresponding reference-scale weights. This mathematical adjustment was exactly equivalent to setting the calibration adjustment of the WIM instruments to a particular value in the field.

Table 1 presents information concerning the performance of the HSWIM system after it had been calibrated by three different moving-vehicle loading techniques involving a total of 60 different trucks. On June 6, 1984, the pavement surfaces surrounding the AX/WHL reference scale were warped transversely to a 3 percent cross-slope (to the left-hand side) just beyond the 10-ft-long approach aprons. The HSWIM transducers were installed in the main lanes of I-10 where the cross-slope was 2 percent to the right-hand side *(3)*. Calibration

TABLE 1 SUMMARY STATISTICS OF WIM WHEEL, AXLE, AXLE-GROUP AND GROSS VEHICLE WEIGHT ESTIMATES COMPARED WITH THE RESPECTIVE AX/WHL SCALE WEIGHTS FOR 60 TRUCKS CROSSING THE HSWIM SCALES (± 50 MPH) AFTER CALIBRATION, JUNE 6, 1984

WEIGHT ESTIMATED	STATISTICAL INFERENCE VALUE	BASIS FOR CALIBRATION OF WIM SYSTEM		
		5 RUNS OF A LOADED 2-AXLE TEST TRUCK	7 DIFFERENT LOADED 5-AXLE (3-S2) TRUCKS	60 DIFFERENT TRUCKS
WHEEL	MEAN WEIGHT, LBS AX/WHL SCALE = 4650	4950	4590	4580
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+9.3 (15.0)	+0.8 (11.2)	0.0 (10.5)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-27.7 to +46.3	-29.0 to +30.6	-27.2 to +27.2
AXLE	MEAN WEIGHT, LBS AX/WHL SCALE = 9300	9910	9180	9170
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+7.5 (9.5)	-0.3 (7.4)	-0.5 (7.4)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-13.3 to +28.3	-19.6 to +18.9	-19.8 to +18.8
AXLE-GROUP	MEAN WEIGHT, LBS AX/WHL SCALE = 13750	14660	13590	13560
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+6.4 (8.2)	-1.4 (6.1)	-1.6 (6.1)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-10.9 to +23.6	-17.5 to +14.7	-17.7 to +14.6
GROSS-VEHICLE	MEAN WEIGHT, LBS AX/WHL SCALE = 39200	41780	38720	38640
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+5.9 (6.6)	-1.8 (3.8)	-2.0 (3.8)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-3.8 to +15.6	-10.8 to +7.2	-10.9 to +7.0

TABLE 2 SUMMARY STATISTICS OF WIM WHEEL, AXLE, AXLE-GROUP, AND GROSS VEHICLE WEIGHT ESTIMATES COMPARED WITH THE RESPECTIVE AX/WHL SCALE WEIGHTS FOR 61 TRUCKS CROSSING THE HSWIM SCALES (± 50 MPH) AFTER CALIBRATION, JUNE 11, 1984

WEIGHT ESTIMATED	STATISTICAL INFERENCE VALUE	BASIS FOR CALIBRATION OF WIM SYSTEM		
		5 RUNS OF A LOADED 2-AXLE TRUCK (2D)	6 DIFFERENT LOADED 5-AXLE (3-S2) TRUCKS	61 DIFFERENT TRUCKS
WHEEL	MEAN WEIGHT, LBS AX/WHL SCALE = 5400	5740	5510	5350
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+7.2 (10.9)	+3.0 (9.0)	0.0 (8.4)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-17.5 to +31.9	-20.3 to +26.3	-22.3 to 22.3
AXLE	MEAN WEIGHT, LBS AX/WHL SCALE = 10800	11470	11010	10690
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+7.2 (9.2)	+2.9 (7.1)	-0.1 (6.6)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-11.8 to +26.2	-15.4 to +21.2	-17.8 to +17.7
AXLE-GROUP	MEAN WEIGHT, LBS AX/WHL SCALE = 17000	18040	17320	16820
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+6.1 (7.8)	+1.8 (5.7)	-1.1 (5.6)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-9.5 to 21.7	-13.1 to +16.8	-15.7 to +13.4
GROSS-VEHICLE	MEAN WEIGHT, LBS AX/WHL SCALE = 49600	52650	50540	49080
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+5.8 (6.4)	+1.6 (4.0)	-1.3 (3.8)
	95% CONFIDENCE RANGE $\hat{\mu} \pm 2\hat{\sigma}$	-3.8 to +15.4	-7.6 to +10.8	-10.6 to +7.6

of the HSWIM scales attempted to make the WIM-estimated weight values agree with the static weights determined on the AX/WHL scale under these conditions. A pronounced improvement in the agreement of the mean weights was made when seven loaded five-axle tractor-semitrailer (3-S2) trucks were used as the basis for calibration compared with five runs of a loaded two-axle single-unit test truck. When differences in the static weights and the WIM weight estimates for all 60 trucks in the data set were taken as the basis for calibration, the resulting mean WIM-estimated weights were virtually the same as those obtained from using the differences from seven loaded 3-S2 trucks as the basis for calibration. The variability in weight differences about the means, as indicated by the 95 percent confidence range, was not affected significantly by the calibration loading technique.

Information about HSWIM weight estimates and corresponding reference-scale weights for 61 trucks on June 11, 1984, is shown in Table 2. The road surface surrounding the

TABLE 3 SUMMARY STATISTICS OF WIM WEIGHT ESTIMATES COMPARED WITH THE RESPECTIVE AX/WHL SCALE WEIGHTS FOR 86 TRUCKS CROSSING THE LSWIM SCALES (< 10 MPH) AFTER CALIBRATION, JULY 6, 1984

WEIGHT ESTIMATED	STATISTICAL INFERENCE VALUE	BASIS FOR CALIBRATION OF WIM SYSTEM		
		STANDARD 1000 LB TEST WEIGHTS	7 DIFFERENT LOADED 5-AXLE (3-S2) TRUCKS	86 DIFFERENT TRUCKS
WHEEL	MEAN WEIGHT, LBS AX/WHL SCALE = 5180	5190	5200	5140
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+1.0 (6.5)	+1.0 (6.0)	0.0 (6.0)
	95% CONFIDENCE RANGE $\hat{\mu} + 2\hat{\sigma}$	-16.4 to +18.4	-15.2 to +17.2	-16.0 to +16.0
AXLE	MEAN WEIGHT, LBS AX/WHL SCALE = 10350	10,390	10350	10290
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+0.9 (4.7)	+0.9 (4.7)	-0.1 to (4.7)
	95% CONFIDENCE RANGE $\hat{\mu} + 2\hat{\sigma}$	-12.3 to +14.1	-12.2 to +14.1	-13.1 to 13.0
AXLE-GROUP	MEAN WEIGHT, LBS AX/WHL SCALE = 15700	15750	15760	15600
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+0.2 (3.9)	0.2 (3.8)	-0.8 (3.9)
	95% CONFIDENCE RANGE $\hat{\mu} + 2\hat{\sigma}$	-10.6 to +10.9	-10.5 to +10.9	-11.4 to +9.8
GROSS-VEHICLE	MEAN WEIGHT, LBS AX/WHL SCALE = 44180	44320	44340	43900
	MEAN OF DIFFERENCES, % (MEAN OF ABSOLUTE DIFFERENCES)	+0.4 (2.6)	0.4 (2.6)	-0.6 (2.6)
	95% CONFIDENCE RANGE $\hat{\mu} + 2\hat{\sigma}$	-6.0 to +6.7	-5.9 to +6.8	-6.8 to +5.7

AX/WHL reference scale had already been leveled with premixed asphalt paving material. A different group of trucks was weighed, but again, a noticeable improvement in the agreement between mean weight values occurred when five-axle tractor-semitrailer (3-S2) trucks were used for calibration loading rather than multiple runs of the same loaded two-axle test truck. Slight improvement over the 3-S2 truck loading resulted from taking all 61 trucks in the data set as the basis for calibration. The range in variability of the weights was slightly less on this day than it was on June 6, 1984, when the road surfaces beyond the reference scale approach slabs were warped to a 3 percent cross-slope.

The information in Table 3 pertains to weight measurements on 86 trucks that were weighed on the low-speed weigh-in-motion (LSWIM) scales on July 6, 1984. On this day, the adverse cross-slope in the pavement surfaces beyond the level approach aprons to the reference scale had been removed and the LSWIM scales had been reinstalled in the leveled surface. Thus, no adverse performance of either the static reference scale or the LSWIM scale can be attributed directly to an uneven surface. It can be seen from the tabulated values that the mean difference in weights from the LSWIM system was 1.0 percent or less for all calibration techniques including dead-weight test blocks. Variability in the percentage differences, as indicated by the 95 percent confidence range, systematically increased from about ±6 percent for gross-vehicle weights to about ±16 percent for wheel weights.

Overall, this comparison indicated that a much better HSWIM or ISWIM system calibration (not included here) was achieved with loaded tractor-semitrailer (3-S2) trucks than with multiple runs of a loaded two-axle (2D) single-unit test truck. The sample of trucks for these data sets contained approximately 60 percent 3-S2 types of trucks, which was representative of the normal kind of truck mix in the traffic stream at the experimental site. As indicated earlier, LSWIM system calibration was best achieved with the static-weight loading technique.

WIM WEIGHT ESTIMATES FOR TWO TRUCK TYPES

Data from the WIM demonstration project that was referenced previously have also been analyzed to study the performance of a high-quality in-pavement WIM system with respect to the consistency of the weight estimates that were produced for two different truck types. These data were taken from the Radian WIM system after the system had been calibrated on site with calibration loading provided by multiple runs of a loaded two-axle, single-unit test truck. Calibration settings on the system were not changed throughout the 2 weeks of data-taking during the project; therefore, the calibration settings in effect at the time when the selected data set was taken are not necessarily the best possible ones. Nevertheless, the data for the two truck types are comparable as the WIM-system operation was the

TABLE 4 SUMMARY STATISTICS OF WIM WEIGHT ESTIMATES COMPARED WITH RESPECTIVE REFERENCE (AX/WHL) SCALE WEIGHTS FOR TRUCKS CROSSING WIM SCALES AT LOW (< 10 MPH), INTERMEDIATE (30 MPH), AND HIGH (50 MPH) SPEEDS. FIVE RUNS OF THE SAME TWO-AXLE SINGLE UNIT

WEIGHT	STATISTICAL INFERENCE VALUES	WEIGHTS FROM WHEELS ON					
		LEFT SIDE			RIGHT SIDE		
		LSWIM	ISWIM	HSWIM	LSWIM	ISWIM	HSWIM
WHEEL	MEAN WIM WEIGHT, lbs	5890	5590	5140	5875	5835	5635
	MEAN REFERENCE-SCALE WEIGHT, lbs	5610	5610	5610	5835	5835	5835
	MEAN OF DIFFERENCES, %	+3.6	-1.6	-7.4	+0.5	-1.1	-2.7
	STANDARD DEVIATION IN DIFFERENCES, ± %	3.8	5.6	6.7	2.9	5.4	4.2
GROSS	MEAN WIM WEIGHT, lbs	11780	11180	10280	11750	11670	11265
	MEAN REFERENCE-SCALE WEIGHT, lbs	11225	11225	11225	11670	11670	11670
	MEAN OF DIFFERENCES, %	+4.9	-0.4	-8.4	+0.7	0.0	-3.4
	STANDARD DEVIATION IN DIFFERENCES, ± %	1.7	4.9	3.3	2.9	4.4	2.8

SINGLE PASS OF 6 DIFFERENT 5-AXLE TRACTOR-SEMITRAILERS

WEIGHT	STATISTICAL INFERENCE VALUES	WEIGHTS FROM WHEELS ON					
		LEFT SIDE			RIGHT SIDE		
		LSWIM	ISWIM	HSWIM	LSWIM	ISWIM	HSWIM
WHEEL	MEAN WIM WEIGHT, lbs	7655	7815	7040	8305	8315	8300
	MEAN REFERENCE-SCALE WEIGHT, lbs	7650	7650	7650	7860	7860	7860
	MEAN OF DIFFERENCES, %	+0.1	+2.0	-8.1	+5.2	+6.0	+6.2
	STANDARD DEVIATION IN DIFFERENCES, ± %	9.3	8.0	7.1	10.5	6.3	9.5
AXLE-GROUP	MEAN WIM WEIGHT, lbs	12755	13025	11730	13840	13860	13835
	MEAN REFERENCE-SCALE WEIGHT, lbs	12750	12750	12750	13100	13100	13100
	MEAN OF DIFFERENCES, %	-0.8	+0.6	-8.8	+3.7	+5.7	+6.8
	STANDARD DEVIATION IN DIFFERENCES, ± %	8.0	7.5	5.5	10.0	5.5	8.3
GROSS	MEAN WIM WEIGHT, lbs	38265	39075	35190	41515	41570	41505
	MEAN REFERENCE-SCALE WEIGHT, lbs	38255	38255	38255	39310	39310	39310
	MEAN OF DIFFERENCES, %	+0.1	+2.2	-8.0	+5.7	+5.7	+5.6
	STANDARD DEVIATION IN DIFFERENCES, ± %	2.5	4.0	2.1	3.6	4.1	4.3

same throughout the session (June 11, 1984). The road surface surrounding the AX/WHL reference scale was level on this day. Judgment about the possible consequences of using only one type of truck for calibration loading can perhaps be improved by studying this data set.

Summary statistical inference values about the relationships among WIM weight estimates and the corresponding static weights from the reference (AX/WHL) scale are presented in Table 4. In computing these values, the weights of wheels on each side of the trucks were considered separately. Each truck passed successively over the HSWIM, ISWIM, and LSWIM transducers in each wheelpath at the approximate speed shown in the table heading before stopping for sequential weighing of the wheels on each axle by the reference scale. An arithmetic mean was calculated for the reference-scale weights and for the WIM-estimated weights for wheels, axle groups, and gross on each side of the truck for each scale. Next, the difference in the reference-scale weight and the WIM-estimated weight was calculated and expressed as a percentage of the reference-scale weight. An arithmetic mean of these differences was then calculated along with the standard deviation and shown in the table. It is pointed out here that the mean of the differences may be numerically different from the difference of the means. The mean of differences, in this analysis, indicates the average amount by percent by which the individual WIM-estimated weights differed from the corresponding reference-scale weights. The standard deviation in differences indicates the percentage range about the mean into which approximately 68 percent of the weight/weight estimate differences would be expected to fall in a normally distributed population of observations. In this data set, the number of observations is too small to test for normality adequately, but other experience with similar, larger samples indicates that the differences tend to be normally distributed. Thus, the magnitude of the standard deviation in differences can be viewed as a measure of the expected variability scatter in the observed differences.

The left-side LSWIM scale indicated slightly higher weight estimates, on average, than the reference scale for the two-axle truck but nearly the same weight estimates, on average, for the six five-axle trucks. Scatter in the weight differences, as

indicated by the standard deviation, is much larger for the six different five-axle trucks than for the two-axle truck. The right-side LSWIM scale gave virtually the same average weights for the two-axle truck but considerably heavier average weight estimates for the five-axle trucks. The pattern and magnitude of scatter are similar to those for the left-side LSWIM values. It is interesting to note that the largest standard deviation (10.5 percent) in the differences was for wheels on the five-axle trucks on the right-side LSWIM scale.

The left-side ISWIM scale produced quite small means of differences for both types of trucks. All values were within ±2 percent. The standard deviation in the differences ranged between 4 and 8 percent. The right-side ISWIM scale, however, had very small means of differences for the two-axle truck, but values of about +6 percent for the five-axle trucks. The standard deviation of the samples ranged between 4.1 and 6.3 percent for all trucks weighed on this scale.

The mean of differences from the left-side HSWIM scale for both truck types ranged between −7.4 and −8.8 percent. This is the most consistent pattern of differences for both truck types in the data set. The standard deviation in the weight differences from this scale also followed a consistent pattern. The right-side HSWIM scale, on average, underestimated weights for the two-axle truck by about 3 percent and overestimated weights for the six different five-axle trucks by about 6 percent. The standard deviation in the differences for the five-axle trucks was nearly double that for the two-axle truck.

In interpreting these observations, it is important to remember that data from three different WIM systems are presented in Table 4. Each system incorporated transducers and associated instrumentation for each wheelpath (left side and right side). These instruments can be adjusted (calibrated) individually to increase or decrease proportionally the magnitude of the weight estimate within a range of settings provided on the instruments. The calibration settings were not optimized for the particular trucks that have been selected for analysis. The road-surface conditions surrounding every transducer might have been slightly different, thereby affecting the dynamic behavior of each truck wheel that crossed the transducer in a different way.

The relative effects of using only one type of truck, say, the two-axle, single-unit, for calibration loading can be appraised by making a rough estimate of the proportional change in the weight estimates for the five-axle tractor-semitrailers that could be expected if the WIM-system calibration were adjusted to make the mean of differences for the two-axle, single-unit wheel weights equal zero. This would result in the left-side LSWIM scale's underweighing the five-axle units by about 4 percent, and would make the right-side LSWIM scale overweigh these units by about 5 percent. The ISWIM scales would tend to overweigh the left side of the five-axle trucks by about 4 percent and the right side by about 7 percent. The left-side HSWIM scales at this site would probably weigh the five-axle units correctly, and the right-side scales would tend to overweigh these units by about 9 percent. These relationships suggest that the dynamic behavior of the wheels on the five-axle, tractor-semitrailer trucks was different from that for the wheels on the two-axle, single-unit truck at the time that the wheel-force sample was taken by the WIM system. The left-side HSWIM scale weight estimates were least affected by the type of truck, and the right-side HSWIM scale weight estimates were most affected. This points out the need to consider each side of the truck separately in calibrating a WIM system on site. The data set also reflects the fact that the trucks observed were not symmetrically loaded side to side (see Mean Reference-Scale Weight, Table 4). The left side of the two-axle, single-unit test truck was nearly 4 percent lighter than the right side, and the left side of the six five-axle, tractor-semitrailers averaged almost 3 percent lighter than the right side when weighed statically on the reference scale.

In summary, this analysis seems to suggest, as does the one presented in the previous section, that the vehicle types used for calibration loading should be proportioned so that they are representative of the mix of truck types that are expected at the WIM site. At least some consideration must be given to whether the calibration-loading trucks should incorporate tandem-axle groups. It appears that trucks with this axle arrangement interact with the WIM system differently from trucks without tandem axles.

COMPUTATION OF CALIBRATION FACTORS

A procedure for calculating a multiplier, or CF, is then developed that can be applied to the wheel force signals in a WIM system to adjust the mean of the expected differences in the WIM weight estimates and the corresponding static weights to zero for a particular site. Differences in WIM weight estimates and measured static weights for a representative sample of vehicle types selected for calibration loading at each site provide the basis for deriving the required CF. A statistical analysis of the wheel weights for a large group of trucks that were selected from the normal traffic stream indicated that there was a significant difference in the loads carried on the left- and right-side wheels of an axle (discussed in the next section). The computational procedure for CFs, therefore, uses left- and right-side wheel-weight data sets separately. Differences can be calculated for wheel weights by using the following equation:

$$D_i = (W_i - W_{o,i})/W_{o,i} \tag{4-1}$$

where

D_i = difference in the individual wheel weight estimated by the WIM system and that measured by the static scale expressed as a fraction of the corresponding wheel weight measured by the static scale,
W_i = wheel weight estimated by the WIM system for observation i, and
$W_{o,i}$ = wheel weight measured by the reference scale for observation i.

The average relative difference is

$$\bar{D} = \frac{1}{n}\sum_{i=1}^{n}[(W_i - W_{o,i})/W_{o,i}] = \frac{1}{n}\sum_{i=1}^{n}\left[\left(\frac{W_i}{W_{o,i}}\right) - 1\right] \tag{4-2}$$

where n = number of observations.

For a given sample of wheel weight data, the value of this average relative difference, for left or right wheels, or both, will fall into one of the following categories:

$\bar{D} = 0$, meaning that it is not necessary to perform an on-site calibration.

$\bar{D} \neq 0$; in this case, on-site calibration is needed. The CFs can be computed from calibration-loading wheel-weight data. Note that CFs may be different for each transducer.

For the second category, a CF can be derived using a set of wheel weight data, as follows. The value of $\bar{D}$ equals the required adjustment to the wheel-weight estimate, a, thus,

$$\bar{D} = \frac{1}{n}\sum_{i=1}^{n}\left[\left(\frac{W_i}{W_{o,i}}\right) - 1\right] = a \tag{4-3}$$

This expression (4-3) can also be stated as

$$\frac{1}{n}\sum_{i=1}^{n}\left(\frac{W_i}{W_{o,i}}\right) = 1 + \bar{D} \tag{4-4}$$

where $\bar{D}$ is not equal to zero (i.e., $\bar{D} = a$).

In order for $\bar{D}$ to fall into the first category previously mentioned (i.e., $\bar{D} = 0$, so that, on the average, WIM-estimated weights will not be different from static weights), the right-hand side of the expression (4-4) must equal 1.0. Both sides of the expression can be multiplied by $1/(1 + \bar{D})$. This puts the expression for $\bar{D}'$, the mean of differences in adjusted weight estimates and corresponding static weights, in the form:

$$\bar{D}' = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{W_i}{W_{o,i}}\right)\left(\frac{1}{1 + \bar{D}}\right) - 1 = 0 \tag{4-5}$$

The multiplier, $1/(1 + \bar{D})$, is the CF that can be applied to WIM wheel-weight estimates to make the average difference in the estimates and the respective static weights equal zero. The CF is simply computed as the reciprocal of the value of $\bar{D}$ (as derived from the data set for each wheel-force transducer, separately) increased by one. This calibration adjustment can be made directly to the force signals from each transducer in the WIM-system instruments or applied to the estimated weights computed by the system.

DISTRIBUTION OF AXLE WEIGHTS ON LEFT- AND RIGHT-SIDE WHEELS

The weight on an axle is usually assumed to be distributed approximately equally between the right and left wheels of the axle; therefore, the gross weight of the truck is assumed to be approximately equally shared by the wheels on the right and left sides of the truck. This assumption is frequently made in analyzing truck-weight data for pavement design and other purposes and is sometimes used for estimating axle loads after the wheels on only one side of a truck have been weighed either statically or dynamically. For example, in Texas, the practice of collecting statistical truck-weight data for many years involved weighing only the right wheels of selected vehicles on a wheel-load weigher and doubling this value for axle weights.

Because the design of pavement and bridge structures is based to a significant extent on the analysis of stress in the structures caused by loads applied to the road surface by the individual wheels of a moving vehicle, wheel-weight data are fundamental. In some pavement design procedures, however, simplifying assumptions that account only for axle loads are made. In order to satisfy the design information needs of all users, a code-specified WIM system should estimate both wheel weights and axle weights for each vehicle. In addition, because the most significant uncontrollable vehicle factor affecting in-motion weighing is tire condition, and because all axle loads are not equally distributed among the wheels of an axle, there is a need for weighing all individual wheels on both sides of a vehicle. Furthermore, weighing on both sides reduces the chance of losing weight data on a truck completely when one of the two WIM system transducers malfunctions. One operable transducer can provide wheel-weight data and serve as a basis for estimating axle loads with some degree of reduced reliability.

Analysis of the wheel-weight data set that was obtained on July 6, 1984, from the special static AX/WHL scale (described previously) indicated that the total weight carried on a tandem axle group (on five-axle tractor-semitrailer trucks of the 3-S2 type) was not equally distributed among all four wheels in the group. Furthermore, this analysis indicated that differences between individual wheel weights and the mean weight of all wheels in the tandem axle sets on the semitrailers were larger than the differences for wheels in the drive-trandem axle groups. By examining this same set of wheel-weight data, a comparison was made of the static wheel weights on the left and right sides of 100 trucks. Data for this comparison are presented graphically in Figure 1.

As shown in Figure 1*a*, individual wheel weights are represented by plotting the left wheel weights against those on the right side of the same axle. This graph clearly indicates that the assumption of equal wheel weights on an axle is not valid, as most of the plotted points do not lie exactly on the 45 degree sloping line of equality. Another form of graphical representation of the data (see Figure 1*b*), indicates the relative difference in the left-wheel weight as a percentage of the right-wheel weight. The right wheel was selected arbitrarily as the reference wheel. It may be noted from Figure 1*b* that, on average, the left-side wheels on these trucks were 3.7 percent heavier than the right-side wheels and that the percent difference in the left-side wheel weight compared with the respective right-side wheel weight on the same axle ranged from 42 percent less to 60 percent more. The results of the Shapiro-Wilk W test *(4, 5)* indicate that these percentage differences can, for statistical analysis purposes, be considered to be normally distributed; therefore, statistically based inferences can be drawn about the probability of wheel weight differences exceeding certain magnitudes due to chance alone. The statistical interpretation of the information shown in Figure 1*b* indicates that, for this population of trucks, 5 percent of the relative differences in the left-side and right-side wheel weights on an axle can be expected to lie outside the −18.1 and +25.4 percent levels. Another statistical test on this data set indicated that the mean value of left-side wheel weights was significantly different from the mean value of right-side wheel weights at a 1 percent confidence level. A greater difference than that observed in the mean values would be expected to occur due to chance alone only once in 100 observations; therefore, it can be concluded that the left-side wheel loads were in fact heavier than the right-side wheel loads for this population of trucks on the average.

Further statistical tests were performed to determine whether there was a statistically significant difference in the average

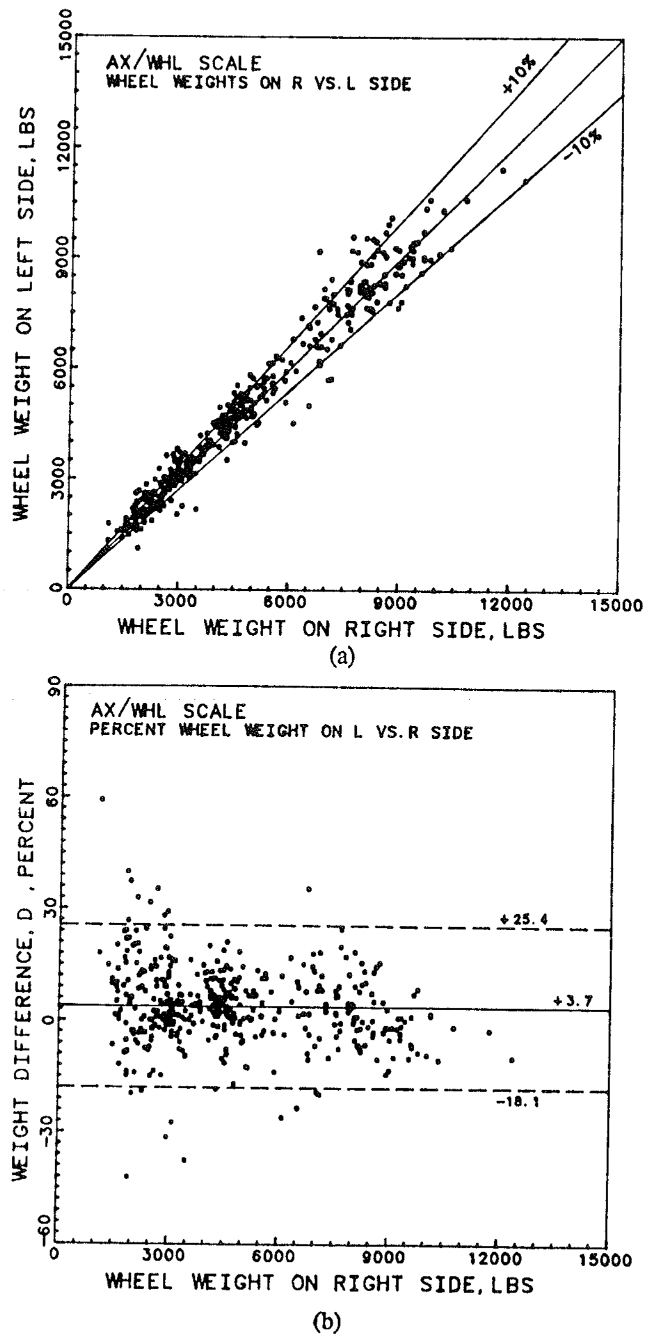

FIGURE 1 ***(a)* Comparison of the weight of the wheels on the same axles on 100 trucks weighed simultaneously on the AX/WHL scale, *(b)* Percent difference in left-side wheels with reference to right-side wheels.**

side-to-side loading of axle groups and in the proportion of the gross vehicle weight carried on the wheels on each side of the trucks. Results of this analysis are summarized in Table 5. The tests indicate that there was a statistically significant difference in the average side-to-side loading of trucks when considering individual axles, axle groups, or gross vehicle weight.

RECOMMENDATIONS

The following recommendations concerning the calibration of WIM systems are based on an evaluation of the previously described data sets and on other experience with the installation and operation of weigh-in-motion systems. Consideration is given to the practicability, safety, and expense of conducting this essential operation on site and under traffic. The need for using wheel weights rather than axle, axle-group, or gross-vehicle weights as the basis for calibration loading has been pointed out.

Before on-site calibration, the inherent limits on the performance capability of each new commercial WIM system configuration should be established under as nearly ideal site conditions as possible via a nationally recognized type-approval program so that each WIM-system user is not required to duplicate this extensive effort. Basic defects or deficiencies in the design or operation of a WIM system cannot be overcome by calibration. On-site calibration can be used, however, to compensate partially for the systematic (biasing) effects of certain local conditions, such as unevenness in the road surface, on WIM-system estimates of vehicle weights.

An accurate determination of the loads that are to be used as the basis for calibrating a WIM system is obviously necessary. If standard dead-weight test blocks are used, these must be of known quality. Likewise, if a force-reaction system (e.g., ram and load cell) is used, the accuracy of the indicated force must be known. If vehicle loading is used, the proportion of the gross-vehicle weight carried by each wheel of the calibration-loading vehicle while its components are in the same attitude as when applying force to the WIM-system transducers must be known. Experience has shown *(3)* that the proportion of gross-vehicle weight that is carried by each individual wheel changes as the wheels move over the road and stop on the scales for weighing and that elevating or lowering the wheel during weighing also causes a load transfer. These effects must be recognized when determining the static wheel weights that will be referenced as the loads used for calibrating a WIM system.

TABLE 5 SUMMARY STATISTICS FOR LEFT AND RIGHT WHEEL WEIGHTS FROM AX/WHL SCALE

STATISTIC	WHEEL WEIGHT	AXLE GROUP WEIGHT	GROSS VEHICLE WEIGHT
Average Right, Lbs	4719	7213	20340
Average Left, Lbs	4841	7398	20863
Mean Difference, Lbs	121.4	185.6	523.4
Standard Deviation of Differences	483.1	642.3	1328.2
Size of Sample	431	282	100
Z-Value	5.22*	4.82*	3.94*
+ Mean Relative Error, %	+3.67	+3.41	+2.73
+ Absolute Mean Relative Error	8.40	6.72	5.37
+ Standard Deviation for Relative Error	10.88	7.97	6.23

* Significant at 95% Confidence Level
\+ Weights on the Left With Reference to Right-Side Weights

The most practical way to measure the individual wheel weights on vehicles that will be used for on-site calibration loading of a WIM system is with wheel-load weighers. These devices are portable and are designed especially to measure wheel loads. Good equipment and proper use of the equipment are both mandatory if accurate measurements are to be obtained. Because all wheels of the vehicle need to be in the same horizontal plane at the time of weighing, multiple (4 or 6 preferred) wheel-load weighers and suitable blocking are required for operating efficiency on a smooth, level surface. Low-profile wheel-load weighers that support dual tires are easier for the vehicle to mount and cause less difficulty when the wheels on the active weighing surfaces are aligned. The lower height also reduces the amount of displacement of vehicle components during the static wheel-weighing process. Alternatively to wheel-load weighers, certain configurations of portable, or fixed, axle-load scales can be used to measure individual wheel loads. It is generally not feasible to weigh individual wheels on a vehicle scale.

LSWIM (< 10 mph) scales should be calibrated against static reference loads. These loads may be applied by standard test weights, a force-reaction system, or the wheels of a standing vehicle. In any case, the range in applied loads should cover the expected-use range (e.g., 1,000–15,000 lb) of interest and include a sufficient number of increments to evaluate the linearity of the system.

ISWIM (±30 mph) and HSWIM (±50 mph) systems should be calibrated with moving-vehicle loads at the time of initial installation and periodically thereafter whenever the local conditions change appreciably. The individual wheel weights of the calibration-loading vehicles must be known. The types of calibration-loading vehicle should be representative of the types of vehicles that are to be weighed at the site. Tandem-axle vehicles seem to interact with the WIM site-specific condition differently from vehicles with only single axles; therefore, the calibration-loading vehicles should have tandem-axle sets if a significant number of tandem-axle weights are to be estimated by the WIM system. If a large proportion of any particular vehicle type is expected (e.g., five-axle, tractor-semitrailers), this vehicle type should be included among the calibration-loading vehicle types. If a single calibration-loading vehicle is to be used, it should have tandem axles. Most heavy axle loads are carried on multiple-axle groups (tandems, triples, and so on); these are loads of critical interest from both the statistical data and the enforcement viewpoint.

A minimum of 30 wheel loads (e.g., 10 passes of three-axle vehicles or 6 passes of five-axle vehicles) should be used as the basis for final adjustment of the mean of the difference in static weight and the corresponding WIM weight estimate to zero for each wheel-force transducer. Preliminary adjustment may be based on fewer loads.

SUMMARY

The importance of on-site calibration of WIM systems has been illustrated by comparing the results of WIM weight estimates made after calibrating the system by various techniques against weights measured on an accurate static reference scale. Mixed truck types were included in the analysis, and high, intermediate, and low speeds were considered. A pronounced improvement in the accuracy with which weights are estimated by the HSWIM and ISWIM systems is achieved when six or seven loaded five-axle, tractor-trailer trucks chosen randomly from the traffic stream are used as the basis for calibration compared with multiple runs of a loaded two-axle, single-unit test truck. The variability in WIM weight estimates is not affected appreciably by the type of moving vehicle used for calibration. A static-weight calibration basis has been found to be adequate for LSWIM calibration.

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