Application of Deflection Testing to Overlay Design: A Case Study

CHERYL ALLEN RICHTER AND LYNNE H. IRWIN

In the fall of 1985, the Engineering Research and Development Bureau (ERDB) of the New York State Department of Transportation (NYSDOT) and the Cornell University Local Roads Program undertook a case study involving the application of the falling weight deflectometer (FWD) to pavement evaluation and overlay design. The site for the case study was a 1-mi section of state highway in the Finger Lakes Region of Central New York State, which had already been scheduled to receive an overlay during the 1986 construction season. Nondestructive pavement testing was conducted in December 1985 and May 1986. Pavement layer moduli were back-calculated from the FWD data using the computer program MODCOMP 2. As a part of the study, a mechanistically based computer program, called PAVMAN, was developed to calculate remaining pavement life and required overlay thickness. The pavement layer moduli determined using MODCOMP 2 were used with the PAVMAN program to estimate the remaining life of the existing pavement and determine the required overlay thickness. The results obtained with PAVMAN compared well with design overlay thicknesses determined using more traditional methods of overlay design (such as engineering judgement and the Asphalt Institute's deflection based method).

In the fall of 1985, the Engineering Research and Development Bureau (ERDB) of the New York State Department of Transportation (NYSDOT) and the Cornell Local Roads Program (CLRP) undertook a joint project concerned with the application of the falling weight deflectometer (FWD) to pavement evaluation and overlay design. The project took the form of a case study involving a 1-mi section of state highway in the Finger Lakes Region of central New York State. The decision to place an overlay at the project site during the 1986 construction season had been made prior to its selection as the site for the case study. The thickness of that overlay had also been decided prior to the selection of the project site.

The case study was the initial phase of a project undertaken by ERDB for the purpose of assessing the capabilities and applications of the falling weight deflectometer and the data it generates. Goals of the case study included:

- To give ERDB personnel an opportunity to become acquainted with the falling weight deflectometer and its use.
- To investigate potential applications of the FWD and the data it generates.
- To investigate the use of mechanistically-based analysis procedures with nondestructive test data.
- To assess the relative merits of different testing approaches (in terms of number and spacing of test points) for routine pavement evaluation.

PROJECT SITE

The site for the case study was a 1-mi section of State Route 96 in Seneca County, New York. Route 96 is a two-lane, rural highway, running basically north-south, with no access control. Traffic data for the project area are given in tables 1 and 2. Because the terrain in the area is very flat and the soils are fairly shallow, drainage in the project area is generally poor.

The pavement in the project area at the time of the testing for the case study consisted of several surface treatments interlayered with two asphalt concrete overlays (nominal combined thickness, three inches) over a bituminous macadam pavement (3-in tar-bound crushed limestone top over a 7-in layer of crushed stone) placed in 1914. The most recent overlay was a 1-in armor coat placed in 1965.

At the time of the deflection testing, the condition of the pavement varied greatly. Some portions of the pavement were intact and exhibited little or no distress. Other portions, particularly sections of the northbound outer wheel path, were badly rutted (rut depths of up to 3½ inches) and/or exhibited severe alligator cracking. Recent pavement condition ratings for the project site are summarized in table 3. The surface and base ratings are on a ten-point scale, where a score of 9 or 10 indicates that the pavement is in excellent condition, while a score less than 6 indicates poor condition, and a score of 1 indicates the worst condition (*1*).

TESTING PROGRAM

Nondestructive Testing

Nondestructive pavement testing for the case study was conducted in December 1985 and May 1986. The testing in December 1985 consisted of FWD and Benkelman beam tests in the inner and outer wheel paths of both lanes. These par-

TABLE 1 TRAFFIC VOLUMES

Year	1960	1970	1973	1976	1977	1980	1986
AADT	1585	1723	1800	1850	2000	1800	3190

C. A. Richter, Strategic Highway Research Program (SHRP), 818 Connecticut Ave., N.W., Washington, D.C. 20006. L. H. Irwin, Cornell University Local Roads Program, Riley Robb Hall, Ithaca, N.Y. 14853.

TABLE 2 VEHICLE CLASSIFICATION DATA (MARCH 1986)

Vehicle Class	Percentage of AADT
Passenger Cars and Pickup Trucks	88
2-Axle, 6-Tire Single Unit Trucks	6
Single Unit Trucks Having 3 or More Axles	1
3-Axle Tractor/Trailer Combinations	0
4-Axle Tractor/Trailer Combinations	1
Tractor/Trailer Combinations With 5 or More Axles	4

allel tests were conducted at 50-ft intervals on two 500-ft sections, one at each end of the project site.

In addition, FWD tests were conducted at intervals of 250 feet over the full 1-mi section in the two outer wheel paths and in the middle of the northbound lane, so that comparisons of the two test strategies (testing at short intervals over subsections of the pavement in question and testing at uniform intervals over the full length of the pavement section) could be made. Each of the test strategies resulted in about 22 tests per mile along each line of test points. In May 1986, only the two 500-ft sections were tested, again with both devices, at 50-ft intervals in the inner and outer wheel paths of both lanes. In all, FWD tests were conducted at 136 points in December 1985 and at 88 points in May 1986.

Three or four FWD drops from one height were conducted at each test point. After discarding data indicative of sensor overflow or other anomalies, the data for each test point were averaged (on a sensor by sensor basis), so that the effects of random error in deflection measurement would be minimized.

Supplementary Testing

In addition to the nondestructive deflection testing, a limited amount of pavement coring, soil boring, and seismographic testing was conducted. The pavement coring and soil boring were done to verify pavement layer thicknesses and to provide samples for laboratory testing. The laboratory tests (including sieve analyses, Proctor densities, sand equivalent tests, and Atterberg limits for the unbound materials and resilient modulus tests, asphalt extractions, and penetration tests for the asphalt concrete) were conducted to characterize the pavement materials for research purposes and would not be needed for routine pavement evaluation. Seismographic testing was conducted to determine the depth to bedrock in the project area, so that the subgrade could be more accurately modeled in subsequent analyses. Seismographic testing would generally not be required unless the depth to bedrock is relatively shallow (less than 30 feet, or so).

TABLE 3 PAVEMENT CONDITION RATINGS

Year	Surface	Base
1981	8	8
1982	7	7
1983	7	7
1984	7	7
1986	6	6

ANALYTICAL METHODS

Back-Calculation of Pavement Layer Moduli

Pavement layer moduli were back-calculated using the computer program MODCOMP 2 (*2*). The assumptions on which MODCOMP 2 is based include the following:

- Pavement layers are homogeneous and isotropic, and extend infinitely in the horizontal plane.
- The deepest pavement layer is semi-infinite.
- The load on the pavement is uniformly distributed over a circular area, and the direction of the force is perpendicular to the pavement surface.
- Full friction exists at the pavement layer interfaces (in other words, there is no lateral slippage of the pavement layers relative to each other).

Using MODCOMP 2, it was possible to obtain reasonable sets of layer moduli for 128 (94 percent) of the December 1985 test points, and 82 (93 percent) of the May 1986 test points. The failure to obtain reasonable solutions for the remaining test points was most likely due to inaccuracies in the pavement model (for example, incorrectly defining the depth to a bedrock layer, the use of a linear model for stress-dependent materials, inaccurate layer thicknesses, boulders at shallow depths in the subgrade, etc.).

Remaining Life and Required Overlay Thickness Calculations

A computer program, called PAVMAN (for PAVement MANagement), was developed to facilitate mechanistically-based analyses of remaining pavement life and required overlay thickness (*3*). PAVMAN was written in the FORTRAN programming language to run on IBM-PC microcomputers.

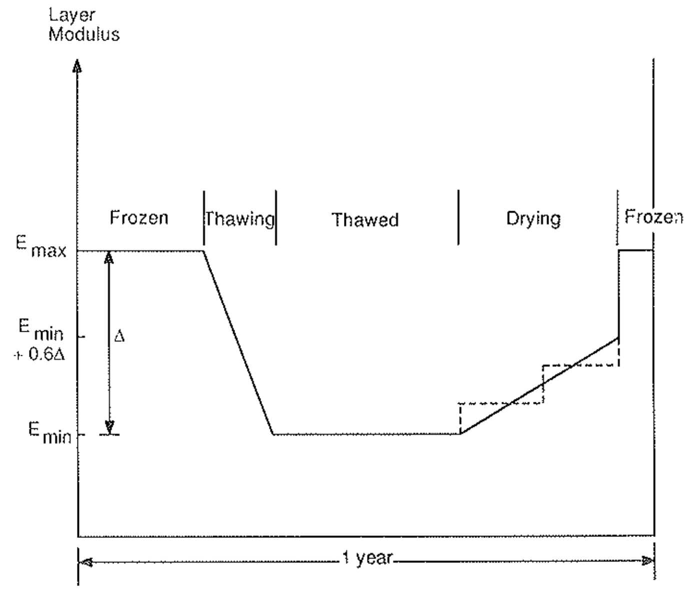

FIGURE 1 Freeze-thaw seasonal variation model.

Selected characteristics of PAVMAN are as follows:

- The pavement is modeled as a layered elastic system, using the same assumptions articulated above for MODCOMP 2.
- Pavement materials may be modeled as being either linear or stress-dependent in their response to load.
- Traffic is considered in terms of an eighteen-kip equivalent single axle load.
- The computer program NELAPAV (*4*) is used as a subroutine to calculate the critical strains induced in the pavement system by a 9-kip dual wheel load.
- Subgrade rutting and surface fatigue are considered as potential failure modes. The fatigue was summed for both the overlay material and the existing surface.
- The revised Shell subgrade strain criteria (*5*) are used to estimate the allowable number of strain repetitions for the subgrade.
- The Shell asphalt concrete fatigue criteria (*6, 7*) are used to estimate the allowable number of strain repetitions for an asphalt concrete surface or overlay.
- The user may select one of three seasonal variation models for each of the pavement layers; a temperature-based model for asphalt concrete, a freeze-thaw seasonal model, or a wet-dry seasonal model. Schematic diagrams of the freeze-thaw model and the wet-dry model are given in figures 1 and 2. Ramps in the layer moduli are modeled as a series of steps (dashed lines in the figures), with each step corresponding to a different season. Since the user defines the magnitude of the seasonal variation as well as the duration of each season, it is thought that these models are sufficiently versatile to model the present state of knowledge of seasonal variation for most circumstances.
- Miner's hypothesis of cumulative fatigue damage (*8*) is applied to sum the damage over the different seasons, and over the life of the pavement.

APPROACHES TO DATA EVALUATION

Having deflection data for roughly one hundred test points in a mile of pavement presents the engineer with an interesting problem—how the data can be used to derive a design overlay thickness (or thicknesses, if it is deemed appropriate to subdivide the section) or an overall remaining life estimate for the section. Traditional deflection based overlay design methods have used a single representative deflection measurement (e.g. mean Benkelman beam deflection plus two standard

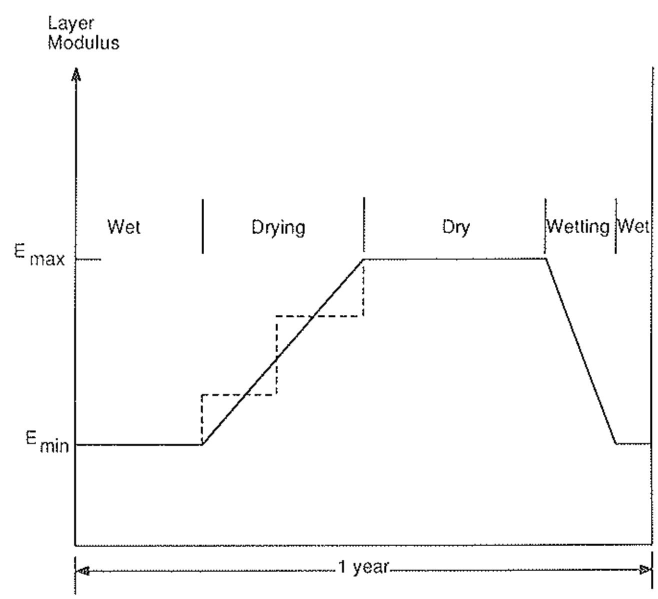

FIGURE 2 Wet-dry seasonal variation model.

deviations, 85th percentile deflection, etc.). At the other extreme, one could determine the required overlay thickness for each individual test point and then decide how to translate the individual overlay thicknesses into a design value.

In deciding between these two approaches to data evaluation, there are some definite tradeoffs. On the one hand, using a representative deflection basin is relatively simple and fast. However, it is not clear that the resulting design overlay thickness will be representative of what the pavement really needs. On the other hand, analyzing the data for each test point individually has the potential to provide much more information about the pavement, but is considerably more time consuming.

Both approaches to the data analysis were used in this case study so that the merits of each could be evaluated more thoroughly. The representative deflection basins used were determined by estimating the 85th percentile deflection (in other words, the deflection that was greater than or equal to 85 percent of the observed deflections) for each sensor in the data set. Separate 85th percentile deflection basins were derived for the test points at 50-ft intervals and those at 250-ft intervals, in the northbound outer wheel path. For both analysis approaches, the remaining life and overlay design analyses were based primarily on the December 1985 deflection data because it was the more extensive data set. Layer moduli derived from the May 1986 deflection data were used to define the magnitude of seasonal variation for the seasonal variation model.

RESULTS

General

Mechanistic Overlay Design

Moduli for the four unknown upper layers (surface, base, subbase, and subgrade) were derived from the deflection basin data using the MODCOMP 2 computer program. The results were evaluated and spurious data were rejected. (It sometimes happens that while MODCOMP 2 will give a good fit of the deflection data, within the specified tolerance, the results are unreasonable and, therefore, must be regarded as spurious.) After the screening process, a set of layer moduli were selected for each test point for use in the overlay calculations. The results were highly variable. A complete summary of the back-calculated moduli is given elsewhere (*3*). Some typical results are given in table 4.

TABLE 4 'TYPICAL' BACK-CALCULATED LAYER MODULI

Layer Depth (Inches)	Modulus (ksi)	Layer Depth (Inches)	Modulus (ksi)	Layer Depth (Inches)	Modulus (ksi)
0-9	380	0-9	668	0-9	588
9-36	6	9-36	28	9-24	94
36-66	30	36-66	7	24-48	5
66-336	250	66-336	250	48-360	250
336-∞	1,250	336- ∞	1,250	360-∞	1,250
0-9	218	0-9	100	0-9	270
9-24	6	9-24	10	9-24	19
24-72	14	24-72	7	24-72	13
72-300	250	72-291	250	72-204	250
300-∞	1,250	291-∞	1,250	204-∞	1,250

Required overlay thickness analyses were conducted for a 15-yr design life using the PAVMAN computer program. The results of the analyses for the individual test points were highly variable, ranging from zero (no overlay needed) to seven inches.

It is believed that this variability is primarily due to variations in the moisture content of the upper subgrade soil. A histogram of the required overlay thickness results is given in figure 3. The test points requiring the greater overlay thicknesses were predominantly in the outer wheel path of the northbound lane but were scattered over the full length of the test section. Many of the larger overlay thicknesses occurred for test points between the two 500-ft sections that were tested at 50-ft intervals (in other words, at points that would not have been tested if only the two 500-ft subsections of the pavement had been tested).

In order to determine the overlay thickness required for a given design life (in this case, fifteen years), PAVMAN calculates (and reports) the remaining life of the pavement for five different trial overlay thicknesses, ranging from zero to

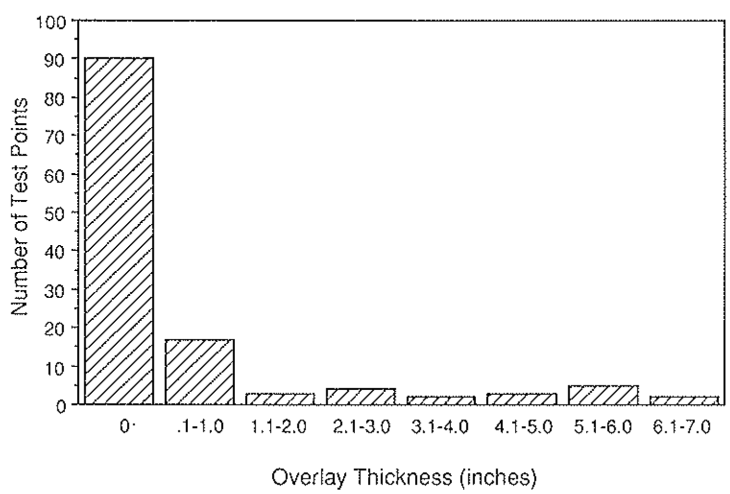

FIGURE 3 Required overlay thickness frequency.

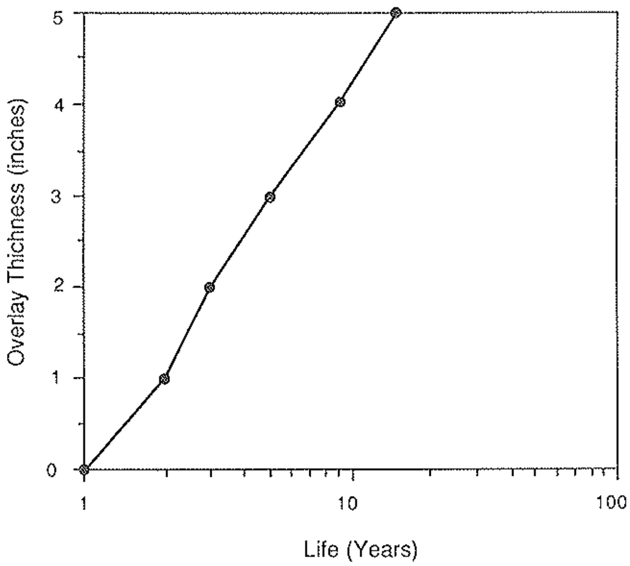

FIGURE 4 Pavement life versus overlay thickness.

a user specified maximum. A typical plot of overlay thickness versus pavement life is given in figure 4. The degree to which such plots are linear in semi-log space appears to depend, to some extent, on the layer that controls the design and whether the controlling layer is the same for all overlay thicknesses.

Eighty-fifth percentile overlay thicknesses were estimated from the individual overlay thickness results for the test points at 50-ft and 250-ft intervals in the northbound outer wheel path, as well as for all of the test points, with results of 2.6, 5.9, and 1.1 inches, respectively. The disparity between the results for the different sets of test points indicates that the number and location of test points selected may significantly affect the results. This indication is further supported by the variability in the individual overlay thickness results.

The required overlay thickness results for the 85th percentile deflection basins, 1.8 inches for the test points at 50-ft intervals, and 6.1 inches for those at 250-ft intervals, compare favorably with the 85th percentile overlay thicknesses derived from the individual test point results for this particular pavement. However, it should not be assumed that this would be true for all pavements. Furthermore, while the degree of variability in the pavement is clearly evident from the individual test point results, it is not at all evident when the representative deflection basin approach is used.

Overlay Design Using the Asphalt Institute Method

The required overlay thickness for the project site was also determined using the Asphalt Institute's deflection based overlay design procedure for comparison (*9*). For these comparisons, the overlay thickness requirement was first determined using the FWD data, so that the comparison would be between different analysis methods based on the same device. Since the Asphalt Institute procedure was developed for use with the Benkelman beam, the Benkelman beam data were used in a subsequent analysis to check the results.

The Asphalt Institute recommends that pavement testing for overlay design be conducted in the outer wheel paths. Therefore, only the data from the outer wheel paths were used in the overlay designs. It was assumed that the deflections measured in the spring of the year were 20 percent lower than the maximum spring values. This assumption was consistent with an assumption made in defining the seasonal variation in layer moduli for the mechanistic analysis. Both the FWD data and the Benkelman beam data were normalized to the 18-kip axle load prescribed in the Asphalt Institute overlay design method.

Although both FWD and Benkelman beam data were available for the project site, it was assumed that the maximum FWD deflection was equal to 50 percent of the Benkelman beam deflection under a similar load. This assumption was somewhat arbitrary, but is in reasonable agreement with the test results for the pavement in question. The assumed correlation was used instead of an exact correlation, so a typical assumption that might be made when only FWD data was available might be tested. Accordingly, the FWD deflections were doubled to approximate Benkelman beam deflections. The traffic volumes and design life used in the Asphalt Institute method were the same as those used for the mechanistically based analysis.

The overlay thicknesses derived from the doubled FWD deflections and the Benkelman beam deflections, for a 15-yr design life, were both approximately 3.8 inches. Thus, for the given pavement and assumptions, it does not matter which device is used, as long as the Benkelman beam deflections are twice the FWD deflections. This will not always be the case.

Interpretation and Application

If it is accepted that it is better to analyze the deflection data on a point by point basis, the question that arises is how does one apply the individual overlay thicknesses once they have been calculated? The strict 85th percentile overlay thickness approach used in the preceding comparisons is one approach, but it has several flaws. First, it does not really make use of the information on the variability in the pavement, and second, a design overlay thickness so derived can be unduly influenced by a few excessively weak test points. To make full use of the information available, the engineer must determine what the required overlay thickness results tell about the pavement.

One interpretation is that individual required overlay thicknesses that are substantially higher than some norm (the mean, median, eighty-fifth percentile value, an arbitrarily selected value, etc.) are indicative of areas that need spot improvements over and above the overlay to be place on the entire pavement. For the purposes of this case study, the following procedure for deriving a design overlay thickness was proposed and used:

1. Calculate the required overlay thickness for each test point.
2. Determine what proportion of the test results yield a negligible required overlay thickness.
3. If the proportion of test points requiring negligible overlay thicknesses is large, consider spot improvements (such as drainage improvements, cut-out and replacement of small sections of pavement, patching, etc.) for the locations that need

work, and use remaining life analyses to determine when the pavement needs to be considered for an overlay. Alternatively, one could determine the overlay thickness required in some given future year.

4. If the number of test points requiring negligible overlay thickness is small, proceed as follows:

(a) Consider the spacing and magnitude of the non-zero required overlay thickness results to determine which points should be considered for spot improvements in addition to the overlay placed on the entire pavement. Spot improvements might include additional overlay thickness, drainage improvements, cut-out and replacement of small sections of pavement, etc..

(b) Since the overlay is being designed to improve those areas that need an overlay, as opposed to those that do not, neglect all zero required overlay thicknesses in subsequent determinations.

(c) Determine the 85th (or other design level) percentile overlay thickness from the set of non-zero overlay thicknesses for test points which have not been selected for spot improvements. This becomes the design overlay thickness. The presumption here is that spot improvements will reduce the required overlay thickness at the spot improvement locations to the level required by the remainder of the pavement.

Table 5 summarizes design overlay thicknesses derived from the data for the inner and outer wheel paths of both lanes, using the procedure outlined above, for the following improvement options:

- No spot improvements are to be made.
- Spot improvements will be made at locations requiring overlay thicknesses of 6 inches or more.
- Spot improvements will be made at locations requiring overlay thicknesses of 5 inches or more.
- Spot improvements will be made at locations requiring overlay thicknesses of 4 inches or more.
- Spot improvements will be made a locations requiring overlay thicknesses of 3 inches or more.

TABLE 5 DESIGN OVERLAY THICKNESSES FOR VARIOUS IMPROVEMENT OPTIONS

Improvement Options[a]	Design Overlay Thickness	Percentage of Pavement Requiring Spot Improvements
1	5.9	0.0
2	5.4	4.7
3	3.8	13.3
4	2.5	15.8
5	2.1	16.3

[a]Numbers refer to the list in the text.

For comparison, the 85th percentile value determined using all of the test points in the wheel paths (including those requiring no overlay) is 1.6 inches.

The data from the testing in the middle of the northbound lane were not included in the determination of the design overlay thickness for two reasons. First, unless some of the data for the northbound lane were omitted, the overlay design would be based more heavily on the northbound lane than the southbound lane because more testing was done in the northbound lane. With the mid-lane data omitted, the two lanes are equally represented in the data set. Second, the relevant portion of the pavement for overlay design purposes (in terms of structural requirements) is the portion of the pavement subjected to significant traffic. Since the portion of the pavement between the wheel paths is not subjected to significant traffic, it should not control or significantly affect the thickness of the overlay required for structural purposes and, therefore, should not be considered in the analysis of required overlay thickness.

In the discussions that follow, it is assumed that spot improvements will be made in the vicinity of all test points having required overlay thicknesses in excess of three inches (Improvement Option 5).

If it is assumed that the necessary spot improvements can be made by placing greater thicknesses of asphalt concrete or by installing underdrains, and that the incremental cost of a 1-in thickness of asphalt concrete is $4.25 per linear foot (based on a pavement width of 20 feet, as per pavement records, an asphalt concrete cost (in place) of $35 per ton, and an asphalt concrete density (in place) of 145 pounds per cubic foot), while underdrains cost $8.00 per linear foot (based on an average trench depth of two feet, a trench width of 18 inches, 6-in perforated steel pipe at $4.65 per linear foot, excavation at $12.00 per cubic yard, filter material (average total depth, 16 inches) at $25 per cubic yard, and backfill (8-in deep, on average) at $4.02 per cubic yard), the installation of underdrains is more economical than placing the required extra thickness of asphalt concrete whenever the underdrains replace more than 1.9 inches of asphalt concrete (whenever the required overlay thickness exceeds the design overlay thickness by more than 1.9 inches).

Under the assumption that spot improvements will be made at all locations requiring overlay thicknesses in excess of three inches, the design (85th percentile) overlay thickness is 2.1 inches. Therefore, the installation of underdrains is the more economical method of making the required spot improvements whenever the required overlay thickness is four inches or more (2.1 + 1.9), and the placement of a thicker overlay is more economical where the required overlay thickness is less than four inches. For the purpose of these comparisons, it assumed that the additional overlay thickness would be placed across the full width of the pavement. Under certain circumstances, it might be possible to taper the overlay so that only the portion of the pavement that needed the additional thickness got the full thickness.

Of the locations that need spot improvements, only one area has a required overlay thickness less than four inches. Therefore, under the given assumptions, with a design overlay thickness of 2.1 inches, the most economical method of rehabilitating the pavement is to place a total overlay thickness of 3.6 inches in that one area and to install underdrains at the other locations needing spot improvements.

Comparison With Other Methods

The thickness of the overlay placed at the project site in the summer of 1986 was determined by NYSDOT engineers using judgment and experience, without the benefit of the nondestructive testing or the subsequent analyses. That overlay consisted of up to 3½ inches of binder material placed to fill in pavement ruts, followed by a 1½-in binder course and a 1-in top course. Thus, the pavement was overlaid with a minimum of 2½ inches of asphalt concrete in areas with minimal rutting and a maximum of 6 inches in severely rutted areas. Spot drainage improvements were made only at the extreme northern end of the project site. Referring to table 5, the 2½-in minimum overlay thickness is comparable to the design overlay thickness determined for the instance where all points needing an overlay 4 or more inches in thickness were subject to spot improvements. If it is assumed that the test points for which high required overlay thicknesses were calculated were in the more severely rutted sections of the pavement, there is reasonable agreement between the required overlay thicknesses calculated for a 15-yr design life and the overlay actually placed.

The 3.8-inch design overlay thickness determined using the Asphalt Institute method is comparable to the design overlay thickness determined by using the PAVMAN program and assuming that spot improvements would be made at points requiring overlays in excess of 5 or 6 inches (see table 5). Thus, if the 3.8-in overlay were placed with no spot improvements, we would expect a higher premature failure rate than if the spot improvements were made, as is assumed in the mechanistically based design.

IMPLICATIONS AND UNANSWERED QUESTIONS

Why Bother?

The reader may well be wondering if mechanistically based pavement analysis and overlay design methods are really worth all the trouble. In the authors' opinion, the answer is absolutely yes, if they are used to their fullest potential. If the decision to place an overlay has already been made, and the year in which it is going to be placed has already been determined, so that the only reason for using the analysis procedure is to decide how thick the overlay should be, then a mechanistically based analysis procedure may not be worth the trouble. However, if the procedure is instead used to determine the optimal timing for a pavement rehabilitation project, and/or the most economical means of rehabilitating a pavement, preferably within the framework of a network-level pavement management system, then mechanistically based pavement analysis techniques have great potential.

With mechanistically based pavement analysis techniques, one can examine the consequences of putting off an overlay for a few more years or model the effects of base stabilization, surface recycling, or improved drainage. One can look at the tradeoffs involved in rehabilitating section A this year instead of section B, or in constructing a new pavement in stages, instead of placing the entire structure needed for the design life at once. In short, mechanistically based pavement analysis and design methods have the potential to be a tremendous tool for optimizing the use of funds in the highway industry.

What's Missing?

While mechanistically based pavement design and analysis methods have tremendous potential, there are several areas in which further research is needed to perfect the methodology. The following sections discuss some of these areas.

Testing Program

The optimal number and spacing of NDT test points for use with mechanistically based pavement evaluation and design methods has not been determined. This case study attempted to address the issue to a limited extent. From the work that was done, it appears that both closely spaced test points over short subsections of the pavement in question and uniformly spaced test points over the full length of the pavement have some advantages and disadvantages. Until further studies can be done to develop reliable criteria, a reasonable compromise would be to test the full length of the pavement in both the inner and outer wheel paths at 250-ft intervals, with the test points in the inner wheel path offset by 125 feet from those in the outer wheel path (in other words, if the first test point in the outer wheel path is at station 0+00, then the first test point in the inner wheel path should be at station 1+25).

Failure Criteria

It is generally held that the critical strains in a pavement structure are the maximum horizontal tensile strain in the surface material and the vertical compressive strain at the top of the subgrade. The criteria used to determine the allowable number of strain repetitions at each of these locations are, as a matter of necessity, empirically derived and, therefore, may not be entirely applicable to materials and circumstances that differ from those for which and under which they were developed. For example, The Shell revised subgrade strain criteria used in the PAVMAN program, as well as many other subgrade strain criteria, were derived from the AASHO Road Test data. Since the AASHO Road Test involved only one subgrade soil and one climate, the use of those criteria for pavements in other areas is an extrapolation that may or may not be valid. Further research is needed to develop more universally applicable strain criteria, for both subgrade and surface materials, and to establish limitations for those that currently exist.

The 18-kip Equivalent Axle Load Question

Frequently, pavement design is based on the 18-kip equivalent single-axle load (ESAL). That is, all vehicle loads are translated to an equivalent number of 18-kip single-axle loads. Like the subgrade strain criteria, most commonly used sets of 18-kip equivalency factors were derived from the AASHO Road Test data (separate factors exist for flexible and rigid pavements). The problem with this is that the number of loads of any given magnitude that is equivalent to an 18-kip ESAL is different for every pavement, so that every time the AASHO equivalency factors are used, an error of unknown magnitude is made. In a mechanistically based analysis, this weakness could be eliminated by treating each vehicle category separately and summing the damage due to each. The cost of doing this would be an increase in computation time. Whether the accuracy gained would be worth the added cost is something that needs to be examined. Alternatively, as microcomputer technology improves, the added cost of considering the damage due to individual vehicle classes may become negligible and thus negate the advantage of using the 18-kip ESAL.

Staged Construction, Alternative Materials, and Rehabilitation Methods, and Network Level Pavement Management

While mechanistically based pavement analyses have the potential to consider staged construction, alternative pavement materials, and alternative rehabilitation methods (stabilized, recycled, or otherwise modified materials, improved drainage, etc.), such analysis alternatives have not necessarily been fully implemented at the project level, let alone at the network level.

The PAVMAN program developed for this case study is a starting point. However, more advanced computer programs with the capability to model staged construction, pavement networks instead of single projects, and rehabilitation methods other than simple overlays, are needed before mechanistically based pavement analysis methods can be used to their fullest potential.

SUMMARY

A case study, in which nondestructive test technology and mechanistically based analysis techniques were applied to the evaluation of required overlay thickness for an existing pavement has been described. A computer program, PAVMAN, developed to facilitate the mechanistic analysis of the pavement, is also described. The design overlay thickness determined using PAVMAN is in reasonable agreement with those derived for the same pavement using the Asphalt Institute's deflection-based overlay design procedure. Actual construction, based on design by experience, also closely matched the results from the mechanistic analysis. Recommendations regarding testing programs for similar activities are made, and areas in which the technology needs further development are discussed.

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The views and opinions expressed herein are those of the authors, and do not represent official views or policies of the New York State Department of Transportation or Cornell University.

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