Evaluation of the 1986 AASHTO Overlay Design Method

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The 1986 AASHTO Guide for Design of Pavement Structures is currently being evaluated in Oregon for use in the design of overlays. This newly revised guide presents two nondestructive methods for determining the strength of existing pavement structures so that the remaining life of the pavement can be evaluated. In the evaluation of the AASHTO overlay design procedure, five project sites around Oregon were selected, including four flexible pavements and one rigid pavement. Deflection measurements were taken using both the Falling Weight Deflectometer and Dynaflect. Cores were also tested to aid in evaluating paving materials. Three backcalculation programs (BISDEF, ELSDEF, MODCOMP2) were used to compute the moduli of the pavements, and the results were used to perform the overlay design. These results were compared with the current ODOT procedure, which is based on tolerable maximum deflection; and considerable inconsistency was observed. Generally, the AASHTO method provided a thinner overlay than the ODOT procedure. The results of this study show that the guide has an important advantage in that pavement strength for each layer can be quantified (NDT method 1) and remaining life may be taken into account. However, further investigation is still required to verify the method of determining existing pavement strength from backcalculation programs and the remaining life of the existing pavement. Therefore, the guide should be used with caution at the present time.

Problem Statement

Currently, the Oregon Department of Transportation (ODOT) uses the California Transportation Department (Caltrans) Procedure with some modifications to design flexible overlays over distressed highway pavements (*1*). The Portland Cement Association (PCA) and American Association of State Highway and Transportation Officials (AASHTO) methods are employed for Portland cement overlays (*2, 3*). Currently, either the Dynaflect or the Falling Weight Deflectometer (FWD) are used to obtain deflections for the flexible overlay design procedure. The maximum surface deflection obtained using the FWD or Dynaflect (converted to an equivalent Benkelman beam deflection) is used in the modified Caltrans method (*4*). For Portland cement concrete overlays, the overlay thickness is determined by subtracting the new design from the effective thickness of the existing pavement (PCA and AASHTO methods).

In both instances, the data generated are insufficient to define accurately the structural adequacy of the existing pavement. In addition, the current procedures do not take into account the remaining life of the existing pavement. To enable designers to make better evaluations on the remaining life of the pavement and provide for more efficient utilization of paving materials, a new overlay design method is needed. The development and use of this new procedure should assist in determining the structural capacity and remaining life.

Purpose

The purpose of this paper is to present an evaluation of the use of the 1986 AASHTO Guidelines (*5*) on selected projects in Oregon. This has included the following steps:

1. Selecting typical project sites for deflection measurements and materials sampling;
2. Laboratory testing materials sampled from each project;
3. Analyzing deflection basin data and developing overlay design recommendations;
4. Discussing results; and
5. Developing appropriate conclusions and recommendations.

1986 AASHTO OVERLAY DESIGN METHOD

Concept

This overlay design procedure is based on the serviceability—traffic and structural capacity—traffic relationships developed at the AASHTO Road Test. Determination of an overlay is accomplished by using a deficiency approach; Figure 1 illustrates seven steps that are generally involved. Of these steps, materials characterization and effective structural capacity analysis require the most effort. Two nondestructive test (NDT) methods are presented in the guide and can be used to analyze the existing pavement structure. They are (1) determination of pavement layer moduli (NDT method 1) and (2) determination of the total structural capacity (NDT method 2). Both methods rely on the use of deflection data generated from a nondestructive testing device.

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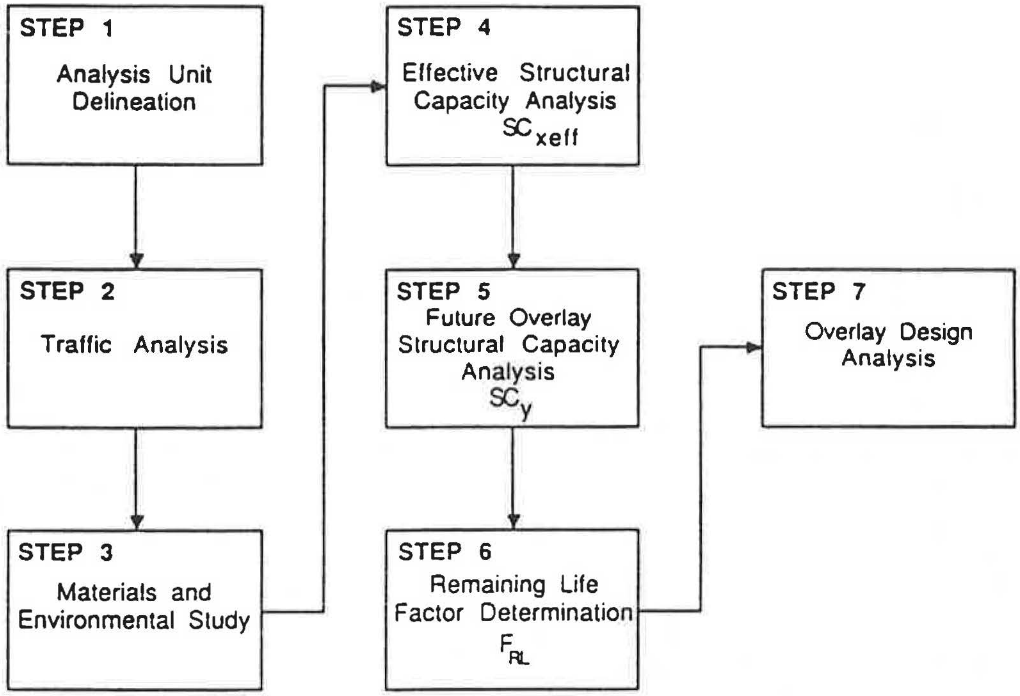

FIGURE 1 Required overlay design steps.

NDT Method 1

NDT method 1 is a technique used to determine the structural capacity of an existing pavement. This technique uses measured deflection basin data from an NDT device to backcalculate the in situ layer elastic moduli, and it is applicable to both flexible and rigid pavements. The fundamental premise of this solution is that a unique set of layer moduli exist such that the theoretically predicted deflection basin is equivalent to the measured deflection basin. To implement this technique, a computer program that backcalculates the elastic modulus for each pavement layer is necessary. The obtained moduli are related to layer coefficients using various charts given in the guide. The structural number is then determined using the equation

$$SN = \Sigma a_i h_i$$

where a_i equals the layer coefficient for each layer and h_i equals the thickness of each layer above subgrade.

NDT Method 2

NDT method 2 is based upon the maximum measured deflection from the dynamic NDT equipment and, as such, does not require a computerized model to backcalculate layer moduli (E_i). With NDT method 2, the maximum measured deflection is used to determine effective pavement structural number (SN_{xeff}) from Burmister's two-layer deflection theory. For a particular pavement structure, the SN_{xeff} value can be determined by a trial-and-error process. This is done by assuming an SN_{xeff} and computing the deflection d_o. If the calculated d_o does not agree with the maximum measured deflection (temperature adjusted), a new SN_{xeff} is assigned. The process is repeated until the calculated deflection matches the maximum measured deflection. A computer program has been developed to solve these equations (*6*).

PROJECTS EVALUATED

Project Descriptions

Field data were collected in the spring of 1987 at five project sites on existing highways in the state of Oregon. Four of the project sites were flexible pavements, and one was a rigid pavement. The age of the projects ranged from 10 to 25 years. Figure 2 shows the location of the project sites, and Figure 3 shows the typical cross sections.

For each of the project sites, data were collected on past and current traffic volumes. The new AASHTO overlay design traffic analysis suggests that two types of data be collected: the cumulative 18k ESAL repetitions until an overlay is placed and the cumulative 18k ESAL expected in the future for the overlay. However, the historic traffic is required only if the traffic method of determining remaining life is used. Table 1 includes a summary of traffic information obtained for each project.

The existing pavement conditions for the five sites varied considerably from one to the other. Two of the test sites (King's Valley Highway and Salem Parkway) did not show any signs of pavement surface distress. The Lancaster Drive site had been overlaid the previous year and, at the time its surface was tested, was in an excellent condition. The Willamina-Salem Highway site showed a considerable amount of cracking, both alligator and longitudinal. The PCC site (Wilsonville-Hubbard Highway) showed a fair amount of cracking in most slabs.

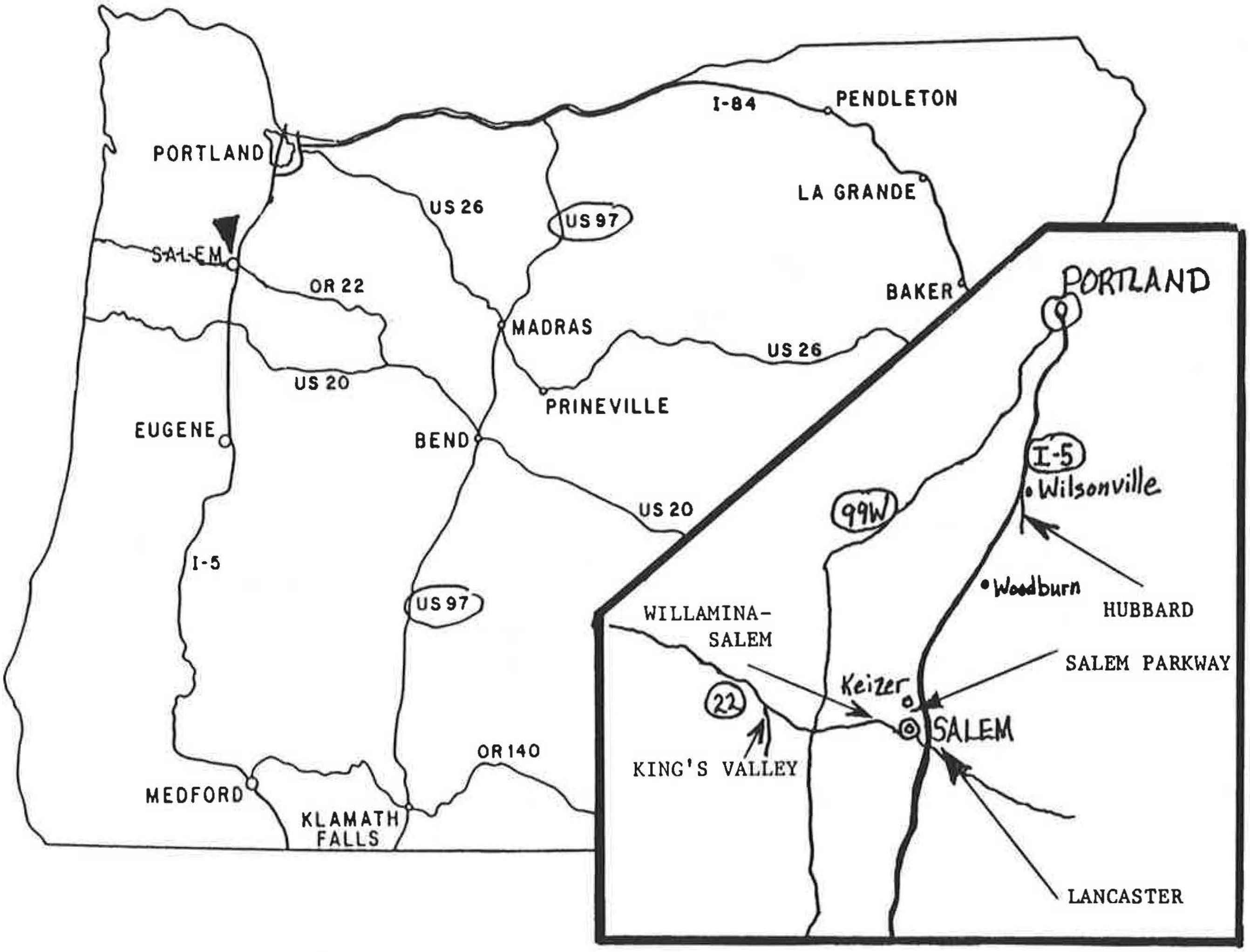

FIGURE 2 Location map of the project sites.

Pavement Deflection Measurements

Pavement surface deflections were measured at 50-ft intervals for 1,000-ft sections for each project. The measurements were taken with the KUAB Falling Weight Deflectometer (FWD) and the Dynaflect, both owned and operated by ODOT. For each site, deflection basin measurements were taken in the outer wheelpath. The FWD data were taken at three load levels and converted to a 9,000-lb load level by simple linear interpolation. The Dynaflect data were measured at a 1,000-lb cyclic load and at a frequency of 8 Hz. The Dynaflect and FWD tests were conducted at the same locations so direct comparisons could be made.

The Dynaflect employs two counter-rotating masses to apply a peak-to-peak dynamic force of 1,000 lb (4.4 kN) at a fixed frequency of 8 Hz. The force is applied to the pavement through the use of two steel wheels 20 in. (50.8 cm) apart, and the deflection basin is measured using five sensors. The spacing of the sensors on this equipment is 1 ft. The ODOT-owned KUAB Falling Weight Deflectometer is trailer-mounted and towed by a ¾-ton van. The impulse force is created by dropping a set of two weights from different heights. By varying the drop height, the load at the pavement surface was varied from 4,900 to 11,300 lb. The two-mass system is used to create a smooth load pulse similar to that created by a moving wheel load (*7, 8*). Surface deflections were measured with four seismic transducers (seismometers) that are lowered automatically with the loading plate and spaced 12 in. apart. Since the FWD can apply a load pulse similar to that produced by a loaded truck, there is no need to correct the determined in situ moduli for stress sensitivity. The load configurations for both the FWD and Dynaflect are shown in Figure 4.

Deflection Results

For each analysis section, the mean and standard deviations of the maximum measured deflection were calculated. Those basins with a maximum deflection that varied by less than the mean or more than 1.5 standard deviations from the mean were discarded. Of the remaining basins, five were randomly selected for further analysis.

In Table 2, the 9,000-lb load level is for the FWD and the 1,000-lb load level, for the Dynaflect. The deflection basins for the FWD were obtained at the 9,000-lb load level by linearly interpolating between the two adjacent load levels.

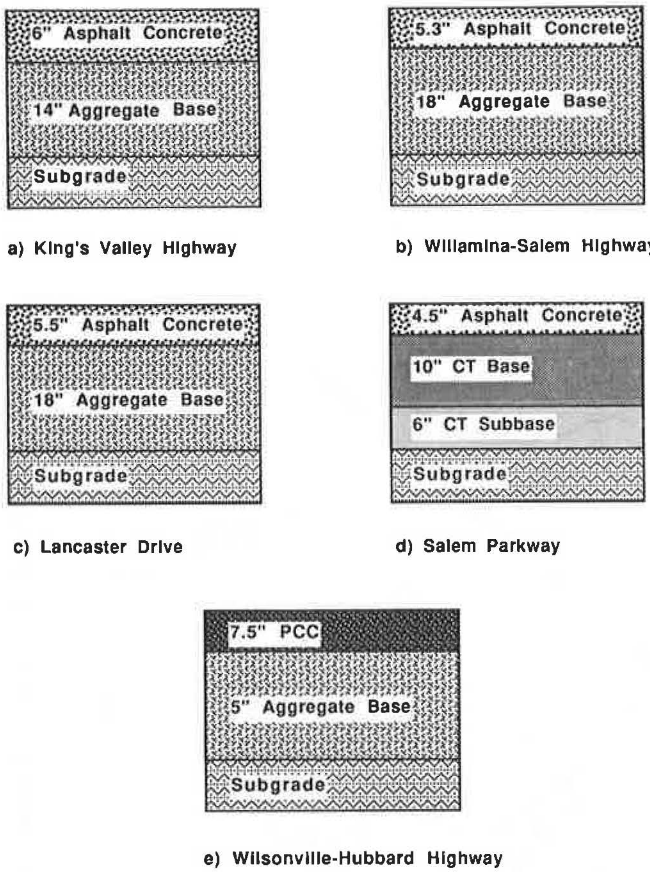

FIGURE 3 Cross sections of pavements analyzed.

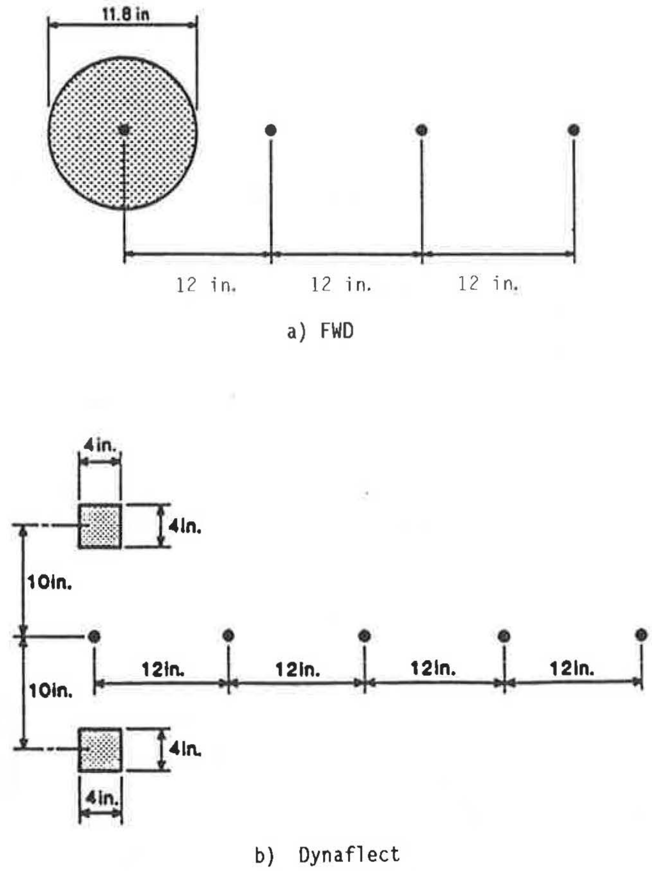

FIGURE 4 Load configuration for both NDT test units.

TABLE 1 SUMMARY OF PROJECT DATA

Project	Cross-Section	Traffic	Pavement Condition
King's Valley Highway	6.0" AC 14.0" Agg. Base Subgrade	4500 ESAL/yr 10 yr TC = 6.0 Cumulative ESAL = $5x10^4$ Future traffic = 33,200	Good surface condition Drainage adequate
Willamina-Salem Highway	5.3" AC 18.0" Agg. Base Subgrade	Current = 173,200 ESAL/yr 10 yr TC = 9.7 Cumulative ESAL = $2x10^6$ Future traffic = 1,876,600	Fair to Poor Longitudinal and alligator cracking in all lanes Evidence of rutting on outside lanes
Lancaster Drive	5.5" AC 18.0" Agg. Base Subgrade	Total Accum. 15 years ~40,000 ESAL/yr 20 yr Future traffic = 1,000,000	Surface condition very good Drainage good
Salem Parkway	4.5" AC 10.0" CTB 6.0" CTS Subgrade	Current = 135,000 ESAL/yr Accumulative = 420,000 ESAL 20 yr Future traffic = 3,200,000	Surface condition and drainage very good
Wilsonville-Hubbard Highway	7.5" PCC 5.0" Agg. Base Subgrade	Current = 115,000 ESAL/yr 10 yr TC = 9.1 Future traffic = 1,097,300	Good to Poor Cracking of slab Erosion of shoulders

Note: TC = Traffic Coefficient = $9.0 \left(\frac{\text{18 kip EAL's}}{10^6}\right)^{0.119}$

TABLE 2 DEFLECTION VALUES FOR THE PROJECTS EVALUATED

Reading Number	Equipment	Load (lbs)	Sensors ($\times 10^{-3}$) in.				
			1	2	3	4	5
King's Valley Highway							
1	FWD	9000	20.9	16.1	10.6	6.31	
	Dynaflect	1000	1.02	0.76	0.47	0.27	0.16
2	FWD	9000	20.76	16.14	10.67	6.25	
	Dynaflect	1000	1.12	0.81	0.49	0.28	0.16
3	FWD	9000	22.17	16.79	10.52	5.72	
	Dynaflect	1000	1.26	0.90	0.51	0.28	0.17
4	FWD	9000	22.19	16.97	10.47	5.77	
	Dynaflect	1000	0.95	0.74	0.47	0.29	0.17
5	FWD	9000	22.27	16.39	9.88	5.02	
	Dynaflect	1000	1.09	0.77	0.43	0.24	0.14
Willamina-Salem Highway							
1	FWD	9000	36.57	22.65	10.39	4.47	
	Dynaflect	1000	1.76	0.98	0.42	0.20	0.12
2	FWD	9000	42.35	25.12	10.62	3.98	
	Dynaflect	1000	2.27	1.17	0.45	0.20	0.10
3	FWD	9000	43.82	27.36	11.18	3.69	
	Dynaflect	1000	2.39	1.09	0.37	0.16	0.10
4	FWD	9000	36.77	22.22	9.50	3.32	
	Dynaflect	1000	1.84	1.03	0.43	0.17	0.08
5	FWD	9000	39.80	24.70	8.95	3.76	
	Dynaflect	1000	1.90	1.07	0.47	0.23	0.14
Lancaster Drive							
1	FWD	9000	26.17	19.80	11.10	7.70	
	Dynaflect	1000	1.55	1.11	0.71	0.46	0.31
2	FWD	9000	27.30	20.30	12.08	7.23	
	Dynaflect	1000	1.65	1.16	0.72	0.44	0.31
3	FWD	9000	26.39	21.13	12.62	7.94	
	Dynaflect	1000	1.65	1.22	0.78	0.47	0.33
4	FWD	9000	26.06	19.86	12.08	7.59	
	Dynaflect	1000	1.36	0.97	0.62	0.39	0.26
5	FWD	9000	27.12	20.72	13.36	8.75	
	Dynaflect	1000	1.44	1.11	0.76	0.51	0.37
Salem Parkway							
1	FWD	9000	5.22	4.47	3.26	3.11	
	Dynaflect	1000	0.41	0.31	0.26	0.21	0.17
2	FWD	9000	4.47	3.36	2.81	2.4	
	Dynaflect	1000	0.35	0.29	0.26	0.27	0.15
3	FWD	9000	5.04	4.09	3.11	2.65	
	Dynaflect	1000	0.39	0.32	0.26	0.21	0.17
4	FWD	9000	5.78	3.99	3.42	2.83	
	Dynaflect	1000	0.46	0.37	0.30	0.23	0.19
5	FWD	9000	4.90	3.77	2.90	2.27	
	Dynaflect	1000	0.47	0.39	0.32	0.27	0.23
Wilsonville-Hubbard Highway							
1	FWD	9000	16.6	13.81	10.96	9.20	
	Dynaflect	1000	0.98	0.91	0.79	0.66	0.54
2	FWD	9000	16.76	13.48	10.47	8.58	
	Dynaflect	1000	1.21	1.14	1.03	0.89	0.76
3	FWD	9000	15.69	13.74	11.25	10.19	
	Dynaflect	1000	1.09	1.06	0.97	0.85	0.72
4	FWD	9000	15.43	12.62	9.64	8.00	
	Dynaflect	1000	1.15	1.12	1.02	0.89	0.75
5	FWD	9000	17.49	15.46	12.43	10.50	
	Dynaflect	1000	1.29	1.20	1.05	0.90	0.74

TABLE 3 RESILIENT MODULUS OF ASPHALT AND PORTLAND CEMENT CONCRETE CORES

Project	M_R @ 74°F (psi)		M_R @ 50°F (psi)	
King's Valley Highway AC	608,000 } T*		1,758,000	
	451,000, 375,000 } M*	Av = 568,000	1,409,000	Av = 1,652,000
	732,000, 673,000 } B*		1,791,000	
Willamina-Salem Highway AC	320,000, 307,000 } **		1,162,000	
	396,000 } **	Av = 346,000	1,369,000	Av = 1,272,000
	334,000, 306,000 } **		1,286,000	
Lancaster Drive AC	264,000, 275,000 } T*		1,045,000	
	336,000, 297,000 } M*	Av = 403,000	801,000	Av = 1,242,000
	843,000 } B*		1,881,000	
Salem Parkway AC	217,000, 200,000 } **	Av = 231,750	760,000	Av = 1,149,000
	257,000, 253,000 } **		1,538,000	
Wilsonville-Hubbard Highway PCC	5,891,300, 4,064,700	Av = 4,977,000	N/A	

*T = Top; B = Bottom; M = Middle.

**Samples tested at two strain levels.

The 9,000-lb load was selected to correspond to the standard axle of 18,000 lb commonly used in the United States.

LABORATORY TESTS

Test Procedures

Resilient modulus laboratory tests (ASTM D4123) were performed on representative asphalt concrete core samples (4-in. diameter). Sample preparation consisted of trimming the cores to a height of approximately 2.5 in. The resilient modulus was determined at test temperatures of 50°F and 74°F using a tensile strain value ranging from 75 to 125 microstrain.

The Portland cement concrete modulus tests were performed on 8-in.-high cylinders with a 4-in. diameter at 74°F. The 4-in. cylinders were tested in compression using three strain gauges attached to the side of the specimen. A strain meter was used to detect the change in strain from changes in electrical resistance in the wire gauges. Strain values were recorded at several levels of load.

Results

The laboratory test results are presented in Table 3. The average modulus obtained was the result of testing the top (T), middle (M), and bottom (B) parts of the AC layer. Figure 5 shows the plot of modulus versus temperature for each of the flexible pavements evaluated.

DETERMINING THE EXISTING PAVEMENT STRENGTH

The structural capacity of the existing pavements was estimated using NDT methods 1 and 2 of the AASHTO Guide (*5*). Three backcalculation programs (BISDEF, ELSDEF, and MODCOMP2) were utilized for NDT method 1. For NDT method 2, calculations were performed by following the procedures described in the AASHTO Guide.

Backcalculation Methods

BISDEF

This computer program was developed by the U.S. Army Corps of Engineers, Waterways Experiment Station (*9, 10*). It uses the deflection basin from NDT results to predict the elastic moduli of up to four pavement layers. This is accomplished by matching a calculated deflection basin to the measured deflection basin.

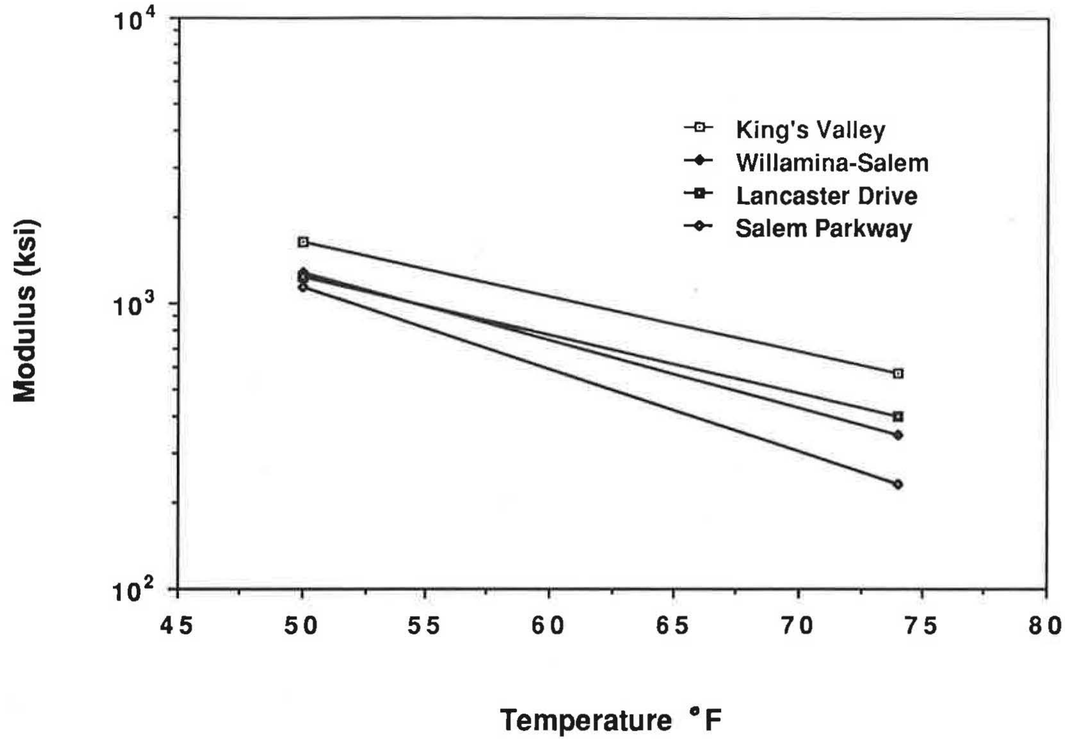

FIGURE 5 Plot of modulus vs. temperature.

To determine the layer moduli, the basic inputs include initial estimates of the elastic layer pavement characteristics as well as deflection basin values. Inputs for each layer include layer thickness, probable modulus range, initial estimate of modulus, and Poisson's ratio. For the deflection basin, the required inputs are load and load radius of an NDT device, deflections at a number of sensor locations, and a maximum acceptable error in deflection matching.

The modulus of any layer may be assigned or computed. If assigned, the value is based on the properties of the material at the time of deflection testing. The number of layers with unknown modulus values cannot exceed the number of measured deflections.

The program is solved using an iterative process that provides the best fit between measured deflection and computed deflection basins. This is done by determining the set of moduli that minimizes the error sum between the computed deflection and measured deflections. BISDEF uses the BISAR program as a subroutine for stress and deflection computations and is capable of handling multiple wheel loads and variable interface friction. BISDEF supports the 8087 or 80287 math coprocessor and runs on IBM-compatible microcomputers.

ELSDEF

This program is a modification of the program BISDEF (*11*). The modification was performed by Brent Rauhut Engineers and uses the computer program (ELSYM5) developed at the University of California at Berkeley (*12*). The input data and output results are basically the same as those of the BISDEF program.

ELSDEF has been compiled with the Microsoft FORTRAN Compiler to run on IBM-compatible microcomputers. Two versions are available: the standard version and an 8087 math coprocessor chip version.

MODCOMP2

This program was developed by Irwin (*13*) of Cornell University. The program utilizes the Chevron elastic layer computer program for determining the stresses, strains, and deflections in the pavement system. As in BISDEF and ELSDEF, there is no closed-form solution for determining layer moduli from surface deflection data. Thus, an iterative approach is used that requires an input of initial or estimated moduli for each layer. The basic iterative process is repeated for each layer (beginning at the bottom) until the agreement between the calculated and measured deflections is within the specified tolerance or until the maximum number of iterations has been reached.

Backcalculation Results

Tables 4 to 8 show the backcalculation results using both the FWD and Dynaflect data on all five project sites. The backcalculation was carried out using the preceding three programs with three different procedures in an attempt to obtain consistent results. Procedure 1 used a fixed surface layer modulus for each project site to determine moduli of the base and subgrade. The surface layer modulus was based on the laboratory test results. Procedure 2 used both a fixed surface

TABLE 4 BACKCALCULATED MODULI (psi) FOR KING'S VALLEY HIGHWAY

a) Procedure 1 - Fixed Surface Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
3	Base	1,000	1,100	N/S**	10,200	9,400	20,700
	Subgrade	60,000	60,000		36,600	32,900	27,600
4	Base	1,000	1,000	3,000	7,400	5,700	10,100
	Subgrade	60,000	60,000	12,000	39,400	40,800	33,100
5	Base	1,000	1,000	N/S	5,300	3,800	6,300
	Subgrade	60,000	60,000		39,500	53,200	41,500
8	Base	1,000	1,000	N/S	14,200	13,600	28,500
	Subgrade	60,000	60,000		32,300	27,500	25,500
18	Base	1,000	1,000	N/S	7,300	5,500	9,500
	Subgrade	60,000	60,000		46,700	51,200	41,500

*Surfacing layer modulus = 1,200,000 psi, determined at 60°F from laboratory tests (Fig. 5).
**N/S = no solution
Value 1,000 is low limit of modulus range for base while 60,000 is high limit for subgrade.

b) Procedure 2 - Fixed Surface and Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
3	Base	9,600	2,700	N/S***	12,300	8,500	N/S
	Subgrade**	10,600	10,600		35,000	35,000	
4	Base	9,300	2,700	N/S	9,800	7,200	N/S
	Subgrade	10,700	10,700		35,000	35,000	
5	Base	6,100	2,300	N/S	8,500	6,100	N/S
	Subgrade	11,700	11,700		32,900	32,900	
8	Base	6,100	2,300	N/S	15,600	9,900	N/S
	Subgrade	11,600	11,600		32,900	32,900	
18	Base	5,000	2,300	N/S	10,000	7,700	N/S
	Subgrade	13,400	13,400		39,900	39,900	

*Surfacing layer modulus = 1,200,000 psi, determined at 60°F from laboratory tests (Fig. 5).
**Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.
***N/S = no solution.

c) Procedure 3 - Fixed Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
3	Surface	472,000	842,300	668,800	1,032,500	1,125,300	3,500,500
	Base	24,700	3,900	8,600	12,200	9,000	4,200
	Subgrade	10,600	10,600	10,600	35,000	35,000	35,000
4	Surface	489,700	853,700	719,400	913,800	973,500	3,228,500
	Base	23,100	3,800	7,600	10,300	8,100	3,400
	Subgrade	10,700	10,700	10,700	35,000	35,000	35,000
5	Surface	427,800	677,400	619,400	687,800	948,100	2,829,900
	Base	16,500	3,800	6,000	9,800	6,800	3,400
	Subgrade	11,700	11,700	11,700	32,900	32,900	32,900
8	Surface	426,100	673,900	655,400	1,225,500	1,465,000	3,717,300
	Base	16,600	3,900	5,500	13,800	8,500	4,600
	Subgrade	11,600	11,600	11,600	32,900	32,900	32,900
18	Surface	379,600	597,100	554,800	756,500	1,062,300	3,250,300
	Base	14,200	3,900	5,600	13,400	8,000	3,700
	Subgrade	13,400	13,400	13,400	39,900	39,900	39,900

*Subgrade modulus was determined using the equation: $E_{sg} = (PS_f)/(rd_r)$.

TABLE 5 BACKCALCULATED MODULI (psi) FOR WILLAMINA-SALEM HIGHWAY

a) Procedure 1 - Fixed Surface Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
4	Base	1,000	1,100	N/S**	8,600	5,200	N/S
	Subgrade*	60,000	60,000		46,100	60,000	
7	Base	1,000	1,000	N/S	5,700	3,800	N/S
	Subgrade	60,000	60,000		34,900	60,000	
8	Base	1,000	1,000	N/S	6,400	3,200	N/S
	Subgrade	60,000	60,000		60,000	60,000	
12	Base	1,000	1,000	N/S	7,100	5,600	N/S
	Subgrade	60,000	60,000		59,000	60,000	
16	Base	1,000	1,000	N/S	7,700	4,300	N/S
	Subgrade	60,000	60,000		41,300	60,000	

*Surfacing layer modulus = 600,000 psi, determined at 68°F from laboratory tests (Fig. 5).
**N/S = no solution
Value 1,000 is low limit of modulus range for base while 60,000 is high limit for subgrade.

b) Procedure 2 - Fixed Surface and Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
4	Base	4,700	2,800	2,800	8,700	6,200	7,300
	Subgrade**	15,000	15,000	15,000	46,600	46,600	46,600
7	Base	3,600	2,300	2,000	6,500	4,400	7,200
	Subgrade	16,900	16,900	16,900	56,000	56,000	56,000
8	Base	3,200	2,100	1,600	6,500	5,300	8,200
	Subgrade	18,200	18,200	18,200	56,000	56,000	56,000
12	Base	4,200	2,700	2,400	6,800	4,900	8,300
	Subgrade	20,200	20,200	20,200	69,900	69,900	69,000
16	Base	4,100	2,600	2,100	8,100	5,700	10,200
	Subgrade	17,900	17,900	17,900	40,000	40,000	40,000

*Surfacing layer modulus = 600,000 psi, determined at 68°F from laboratory tests (Fig. 5).
**Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.

c) Procedure 3 - Fixed Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
4	Surface	199,600	284,300	241,400	203,500	392,600	2,555,600
	Base	8,000	4,000	4,800	10,300	7,300	4,300
	Subgrade	15,000	15,000	15,000	46,600	46,600	46,600
7	Surface	168,500	219,000	214,100	147,500	307,500	1,965,600
	Base	5,900	3,300	2,600	5,100	5,100	3,300
	Subgrade	16,900	16,900	16,900	56,200	56,000	56,000
8	Surface	174,900	215,300	224,200	121,000	251,200	1,706,500
	Base	4,900	2,900	3,400	6,000	7,000	4,300
	Subgrade	18,200	18,200	18,200	56,000	56,000	56,000
12	Surface	203,000	259,600	235,100	279,700	359,700	2,557,000
	Base	6,400	3,600	4,200	8,400	5,700	3,200
	Subgrade	20,200	20,200	20,200	69,900	69,900	69,900
16	Surface	165,900	211,100	236,700	194,900	381,300	2,378,000
	Base	6,500	3,700	3,600	9,900	6,700	4,000
	Subgrade	17,900	17,900	17,900	40,000	40,000	40,000

*Subgrade modulus was determined using the equation: $E_{sg} = (PS_f)/(rd_r)$.

TABLE 6 BACKCALCULATED MODULI (psi) FOR LANCASTER DRIVE

a) Procedure 1 - Fixed Surface Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
2	Base	2,100	1,900	2,000	11,700	17,900	17,800
	Subgrade*	20,800	20,600	19,800	20,400	4,000	17,500
3	Base	1,700	1,400	1,300	9,300	11,700	13,400
	Subgrade	60,000	51,200	86,400	21,800	4,000	19,100
14	Base	1,800	1,500	N/S**	9,300	11,700	13,300
	Subgrade	33,700	30,100		20,100	4,400	17,500
17	Base	2,100	1,800	1,800	14,400	25,600	22,600
	Subgrade	31,600	21,700	22,900	23,300	4,100	19,700
19	Base	2,500	2,300	2,300	18,800	21,400	25,100
	Subgrade	17,400	10,800	13,300	16,500	12,800	14,700

*Surfacing layer modulus = 1,000,000 psi, determined at 57°F from laboratory tests (Fig. 5).
**N/S = no solution; Value 60,000 is high limit of modulus range for subgrade.

b) Procedure 2 - Fixed Surface and Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
2	Base	10,600	4,200	5,100	14,500	10,400	N/S***
	Subgrade**	8,700	8,700	8,700	18,100	18,100	
3	Base	7,400	3,400	4,100	12,700	9,400	N/S
	Subgrade	9,300	9,300	9,300	18,100	18,100	
14	Base	8,600	3,400	4,100	12,500	9,000	N/S
	Subgrade	8,500	8,500	8,500	17,000	17,000	
17	Base	9,400	3,900	5,000	16,400	12,300	N/S
	Subgrade	8,800	8,800	8,800	21,500	21,500	
19	Base	10,000	3,800	5,600	23,100	14,500	N/S
	Subgrade	7,700	7,700	7,700	15,100	15,100	

*Surfacing layer modulus = 1,000,000 psi, determined at 57°F from laboratory tests (Fig. 5).
**Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.
***N/S = no solution

c) Procedure 3 - Fixed Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
2	Surface	287,000	647,100	585,300	383,400	987,300	N/S**
	Base	26,500	6,200	8,300	22,000	10,400	
	Subgrade*	8,700	8,700	8,700	18,100	18,100	
3	Surface	312,000	635,900	537,200	249,300	1,139,600	N/S
	Base	19,300	4,900	7,100	21,700	8,700	
	Subgrade	9,300	9,300	9,300	18,100	18,100	
14	Surface	405,300	765,100	865,000	477,200	882,500	N/S
	Base	18,700	4,600	4,800	17,500	9,600	
	Subgrade	8,500	8,500	8,500	17,000	17,000	
17	Surface	335,800	734,200	621,900	490,200	1,077,000	N/S
	Base	23,200	5,100	7,700	22,200	11,600	
	Subgrade	8,800	8,800	8,800	21,500	21,500	
19	Surface	390,800	834,800	567,500	692,600	1,398,500	N/S
	Base	21,600	4,300	9,200	33,700	11,700	
	Subgrade	7,700	7,700	7,700	15,100	15,100	

*Subgrade modulus was determined using the equation: $E_{sg} = (PS_f)/(rd_r)$.
**N/S = no solution

TABLE 7 BACKCALCULATED MODULI (psi) FOR SALEM PARKWAY

a) Procedure 1 - Fixed Surface Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
7	CT Base/Subbase	1,020,000	501,200	365,300	523,000	1,497,300	183,000
	Subgrade*	18,600	15,300	19,100	34,300	14,100	35,600
9	CT Base/Subbase	690,900	681,800	255,400	439,600	981,200	154,100
	Subgrade	23,400	16,500	31,300	35,100	14,600	36,300
12	CT Base/Subbase	805,200	508,700	299,300	543,900	1,035,500	222,700
	Subgrade	24,600	12,300	23,400	35,000	14,400	34,800
15	CT Base/Subbase	727,700	409,600	459,400	400,800	896,100	194,900
	Subgrade	19,700	14,500	20,800	31,300	14,800	29,900
17	CT Base/Subbase	697,400	491,800	250,600	564,400	1,240,600	200,300
	Subgrade	27,200	13,700	28,200	25,800	10,100	27,700

*Surfacing layer modulus = 500,000 psi, determined at 67°F from laboratory tests (Fig. 5).

b) Procedure 2 - Fixed Surface and Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
7	CT Base/Subbase	751,300	239,000	260,800	585,600	420,100	214,600
	Subgrade*	21,600	21,600	21,600	32,900	32,900	32,900
9	CT Base/Subbase	577,000	207,100	229,800	516,000	375,900	184,400
	Subgrade	32,600	32,600	32,600	33,000	33,000	33,000
12	CT Base/Subbase	671,500	223,700	240,000	604,700	404,400	249,700
	Subgrade	25,400	25,400	25,400	33,000	33,000	33,000
15	CT Base/Subbase	581,800	189,600	314,000	463,800	299,200	200,900
	Subgrade	23,800	23,800	23,800	29,500	29,500	29,500
17	CT Base/Subbase	600,400	212,600	220,400	663,300	340,500	262,300
	Subgrade	29,600	29,600	29,600	24,400	24,400	24,400

*Surfacing layer modulus = 500,000 psi, determined at 67°F from laboratory tests (Fig. 5).
**Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.

c) Procedure 3 - Fixed Subgrade Modulus

Location Identification	Layer*	FWD BISDEF	FWD ELSDEF	FWD MODCOMP2	Dynaflect BISDEF	Dynaflect ELSDEF	Dynaflect MODCOMP2
7	Surface	5,813,400	7,420,300	8,224,900	3,459,500	9,201,800	6,264,500
	CT Base/Subbase	273,400	100,000	130,200	405,600	100,000	125,000
	Subgrade*	21,600	21,600	21,600	32,900	32,900	32,900
9	Surface	919,500	3,387,000	1,762,400	2,549,700	8,157,200	6,360,600
	CT Base/Subbase	476,600	100,000	190,900	370,800	100,000	102,100
	Subgrade	32,600	32,600	32,600	33,000	33,000	33,000
12	Surface	5,127,600	4,224,000	4,807,000	590,117	9,467,400	6,378,500
	CT Base/Subbase	252,800	100,000	153,900	925,800	100,000	147,800
	Subgrade	25,400	25,400	25,400	33,000	33,000	33,000
15	Surface	510,400	3,111,700	526,900	1,030,600	9,246,700	6,673,100
	CT Base/Subbase	556,100	100,000	311,100	609,900	109,000	109,100
	Subgrade	23,800	23,800	23,800	29,500	29,500	29,500
17	Surface	4,408,300	3,722,200	3,234,500	1,673,100	9,186,500	4,106,600
	CT Base/Subbase	230,100	100,000	162,300	608,400	100,000	174,000
	Subgrade	29,600	29,600	29,600	24,400	24,400	24,400

*Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.
Value 100,000 is the low limit of modulus range for the CT base/subbase.

TABLE 8 BACKCALCULATED MODULI (psi) FOR WILSONVILLE-HUBBARD HIGHWAY

a) Procedure 1 - Fixed Surface Modulus

Location Identification	Layer*	FWD			Dynaflect		
		BISDEF	ELSDEF	MODCOMP2	BISDEF	ELSDEF	MODCOMP2
1	Base	1,000	1,000	N/S**	1,000	106,900	N/S
	Subgrade	5,900	3,900		11,100	6,200	
2	Base	1,000	1,000	N/S	1,000	128,600	N/S
	Subgrade	6,400	3,900		7,700	4,000	
7	Base	1,000	1,000	N/S	2,600	215,000	N/S
	Subgrade	5,700	3,800		7,900	4,100	
13	Base	1,000	1,000	N/S	1,000	185,700	24,200
	Subgrade	6,900	4,500		8,100	3,900	7,100
14	Base	1,000	1,000	N/S	1,000	130,100	N/S
	Subgrade	5,000	3,200		7,600	4,800	

*Surfacing layer modulus = 4,977,000 psi, determined from laboratory test.
**N/S = no solution; Value 1,000 is low limit of modulus range for base.

b) Procedure 2 - Fixed Surface and Subgrade Modulus

Location Identification	Layer*	FWD			Dynaflect		
		BISDEF	ELSDEF	MODCOMP2	BISDEF	ELSDEF	MODCOMP2
1	Base	1,000	1,000	N/S***	2,700	1,000	1,600
	Subgrade**	7,300	7,300		10,400	10,400	10,400
2	Base	1,000	1,000	<1,000	2,100	1,000	<1,000
	Subgrade	7,800	7,800	7,800	7,400	7,400	7,400
7	Base	1,000	1,000	<1,000	1,000	1,000	4,900
	Subgrade	6,600	6,600	6,600	7,800	7,800	7,800
13	Base	1,000	1,000	<1,000	1,000	1,000	2,200
	Subgrade	8,400	8,400	8,400	7,500	7,500	7,500
14	Base	1,000	1,000	N/S	1,000	1,000	<1,000
	Subgrade	6,400	6,400		7,600	7,600	7,600

*Surfacing layer modulus = 4,977,000 psi, determined from laboratory test.
**Subgrade modulus was determined using the equation: $E_{sg} = PS_f)/(rd_r)$.
***N/S = no solution; Value of 1,000 is low limit of modulus range for base

c) Procedure 3 - Fixed Subgrade Modulus

Location Identification	Layer	FWD			Dynaflect		
		BISDEF	ELSDEF	MODCOMP2	BISDEF	ELSDEF	MODCOMP2
1	Surface	424,100	1,634,700	389,000	1,944,000	4,436,400	1,255,700
	Base	354,200	1,000	343,700	242,900	1,000	502,000
	Subgrade*	7,300	7,300	7,300	10,400	10,400	10,400
2	Surface	171,500	1,548,700	297,600	3,237,100	3,598,900	817,900
	Base	1,028,600	1,000	390,700	21,000	1,000	735,600
	Subgrade	7,800	7,800	7,800	7,400	7,400	7,400
7	Surface	595,700	1,979,200	676,300	4,982,600	4,056,400	906,500
	Base	272,600	1,000	340,900	2,200	1,000	1,124,200
	Subgrade	6,600	6,600	6,600	7,800	7,800	7,800
13	Surface	345,700	1,733,800	393,700	3,750,100	3,757,200	877,900
	Base	582,000	1,000	314,200	3,400	1,000	479,500
	Subgrade	8,400	8,400	8,400	7,500	7,500	7,500
14	Surface	810,900	1,547,700	858,200	2,631,800	3,167,700	872,900
	Base	159,000	1,000	131,600	24,100	1,000	466,000
	Subgrade	6,400	6,400	6,400	7,600	7,600	7,600

*Subgrade modulus was determined using the equation: $E_{sg} = (PS_f)/(rd_r)$.

layer modulus and a preestimated subgrade modulus to solve for the modulus of the base layer. The subgrade modulus was determined using the following AASHTO equation (*5*):

$$E_{sg} = (PS_f)/(d_r r)$$

where

- E_{sg} = in situ subgrade modulus of elasticity (psi);
- P = dynamic load of NDT device;
- d_r = measured NDT deflection (mils) at a radial distance (r) from the plate load center;
- r = radial distance (in.) from the plate load center; and
- S_f = subgrade modulus prediction factor, which is a function of radius of NDT load plate, Poisson's ratio, and effective thickness of the pavement.

Procedure 3 used the preestimated subgrade modulus alone to solve for the surface and base layer moduli. Procedure 4 used no fixed value. With this procedure, all layers were considered as unknowns for the program to determine moduli. Since three variables were involved, the computing time was significantly increased and the calculated moduli were also subject to variation with the seed moduli. Because of inconsistent results obtained using this procedure, no further discussion is presented.

Procedure 1

Using this procedure, the surfacing modulus for each project was determined from laboratory tests and used as a fixed value in the backcalculation. For Salem Parkway, base and subbase were treated as one layer, thus eliminating one variable for determining modulus. The results from three programs using FWD data indicate that the King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive sites have a weak base layer. On the other hand, using Dynaflect data, a consistently higher modulus for the base layer was found. Results from BISDEF and ELSDEF are relatively close using both FWD and Dynaflect data. MODCOMP2 provided no solution in several cases. Results from three programs using the same NDT device are generally close.

Salem Parkway has a cement-treated base/subbase. Results from three programs reflect this fact. However, the backcalculated modulus values vary for each program. With FWD data, results from BISDEF are higher than those of both ELSDEF and MODCOMP2. With Dynaflect data, ELSDEF presents the highest modulus values among three programs. In all cases, MODCOMP2 provides the lowest values. Subgrade modulus values calculated from BISDEF and MODCOMP2 are relatively close for each NDT testing device, while ELSDEF gives a consistent lower modulus value using both FWD and Dynaflect data.

Wilsonville-Hubbard Highway is a PCC pavement. Its surfacing layer modulus is about 5 million psi as tested in the laboratory. When fixing this value and backcalculating the other two layer moduli, the program BISDEF predicts a very weak base using both FWD and Dynaflect data; ELSDEF gives different solutions using different NDT device data; and MODCOMP2 fails to provide answers in most cases.

Procedure 2

This procedure used two known moduli to determine the third unknown modulus. The surfacing modulus was determined from the laboratory test, while the subgrade modulus was estimated using the AASHTO equation. Since only one variable (base) is defined, the difference, that of backcalculated moduli using different programs, can easily be seen. For all four flexible pavements, the program BISDEF presents a consistently higher modulus than ELSDEF and MODCOMP2. The results of this method show many similarities with those of procedure 1. A weak base at the King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive project sites is indicated. A similar trend at Salem Parkway is also noted. For the PCC pavement at Wilsonville-Hubbard Highway, a very weak base layer is identified by all three programs using both FWD and Dynaflect data. Again, MODCOMP2 failed to give a solution in some test locations.

Procedure 3

The third procedure used an estimated subgrade modulus as a fixed input to solve for surface and base moduli. The results are presented in Tables 4c to 8c. Although the backcalculated moduli vary for each program, the results from each individual program are fairly close for the projects at King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive. As would be expected, Willamina-Salem Highway would have the lowest modulus values since it had the highest measured deflection at NDT device load center, and King's Valley Highway would have high modulus because of its smaller deflection readings, while Lancaster Drive would stand in among these three project sites. The backcalculated results reflect this phenomenon very well. For Willamina-Salem Highway using BISDEF results, the average modulus for the surface is about 180 ksi. The average surface modulus for King's Valley Highway is close to 440 ksi and, for Lancaster Drive, approximately 350 ksi. The backcalculated moduli for the base layers also seem reasonable. Values are generally uniform, with BISDEF giving a little higher modulus. For the cement-treated base/subbase project at Salem Parkway, the three programs give inconsistent results, as can be seen in Table 7c. This fact is also reflected in the Wilsonville-Hubbard Highway, which is a PCC pavement. It is therefore difficult to make a general prediction of pavement strength on these two projects based on the backcalculated moduli using procedure 3.

Existing Pavement Structural Capacity

The structural capacity of the existing pavements was determined using the backcalculation results. For NDT method 1, the SN_{xeff} values for each test location were computed for both the FWD and Dynaflect using the backcalculated results from the BISDEF program. For King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive, backcalculated moduli from procedure 3 were used to determine the SN_{xeff} since they seemed more reasonable compared with the other two procedures. For Salem-Parkway and Wilsonville-Hubbard Highway, procedure 1 results were used because of

TABLE 9 CALCULATED SN_{xeff} USING BISDEF RESULTS

		FWD				Dynaflect			
		NDT #1		NDT #2		NDT #1		NDT #2	
Project Site	Layer	M_R (ksi)*	SC_{xeff}	M_R	SC_{xeff}	M_R	SC_{xeff}	M_R	SC_{xeff}
King's Valley Highway	Surface	439.0	3.84	-	3.70	923.2	3.17	-	8.15
	Base	19.0		-		11.9		-	
	Subgrade	11.6		11.6		35.1		35.1	
Willamina-Salem Highway	Surface	182.4	1.67	-	2.92	189.3	1.75	-	7.26
	Base	6.3		-		7.9		-	
	Subgrade	17.6		17.6		53.7		53.7	
Lancaster Drive	Surface	346.2	3.96	-	3.99	458.5	4.33	-	8.12
	Base	21.9		-		23.4		-	
	Subgrade	8.6		8.6		18.0		18.0	
Salem Parkway	Surface	500.0	5.34	-	6.95	500.0	3.80	-	12.46
	Base	788.2		-		494.3		-	
	Subgrade	22.7		26.3		32.3		30.2	
Wilsonville-Hubbard Highway	Surface	4977.0	6.00	-	6.00**	4977.0	6.00	-	6.00
	Base	1.0		-		1.3		-	
	Subgrade	6.0		7.3		8.5		8.1	

*Average values
**NDT method 2 is not applicable for the evaluation of rigid pavement systems. With this method, structural capacity is expressed in terms of the PCC layer and not the other layers.

inconsistent results of procedure 3 and a good agreement between procedures 1 and 2. The layer coefficients for the surface and base were determined based on the modulus values from the backcalculation.

For NDT method 2, the SN_{xeff} values were determined using the procedures described previously, while the subgrade modulus was estimated using the AASHTO equation. The results for both methods are presented in Table 9.

The results generally indicate the following:

1. A fair agreement in calculating SN_{xeff} between NDT methods 1 and 2 was found using FWD data.
2. While using Dynaflect data, the NDT method 2 results in much higher SN_{xeff} values than NDT method 1 and also higher than those obtained using FWD data.
3. A maximum surface layer coefficient of 0.44 was used for asphalt concrete pavements. This may not reflect the true structural capacity of those surface layers with high modulus.

OVERLAY DESIGN

AASHTO Method

With the AASHTO method, overlay design was performed based upon the existing pavement structural capacity (SN_{xeff}) and future traffic applications (W_{18}). For each project, SN_{xeff} values determined previously were used to estimate the remaining life of the existing pavement and, consequently, the thickness design. In determining the future overlay structural capacity (SC_y), a 90 percent reliability level (R) was chosen for the Willamina-Salem Highway and Salem Parkway projects. An 80 percent reliability was selected for King's Valley Highway, Lancaster Drive, and Wilsonville-Hubbard Highway. The overall standard deviation (S_o) was selected to be 0.35 for all five projects. The design serviceability loss (DSL) was set at 2.0 (4.2–2.2). The selection of the preceding values was primarily based on the functional class and location of the facility and the projected level of usage.

Knowing the future traffic (W_{18}), reliability level (R), overall standard deviation (S_o), design serviceability loss (DSL), and subgrade modulus (M_R), the structural number (SN_y) was determined.

The remaining life of the existing pavement (R_{LX}) was estimated using the NDT approach. The advantage of this method is that historical traffic data are not required. Using this approach, the existing pavement condition is related to its initial structural capacity by a condition factor, C_x. R_{LX} is a function of the value for C_x. The remaining life of overlaid pavements (R_{LY}) was calculated based upon the projected future traffic applications and the ultimate number of repetitions to failure. The failure serviceability level (P_f) was set at 2.0 for all five projects. After determining both R_{LX} and R_{LY}, the remaining life factor, F_{RL}, was estimated.

The required flexible overlay structural number, SN_{OL}, is a function of the structural capacity of the existing pavement (SN_{xeff}), the overlaid pavement (SN_y), and the remaining life factor (F_{RL}). If this value is less than or equal to zero, no overlay is required. The thickness of an overlay is determined by dividing the SN_{OL} by the layer coefficient of the surfacing material. For the five projects, the thickness of a flexible overlay was determined assuming a layer coefficient of 0.44 for the asphalt concrete. Summaries for both NDT method 1 and NDT method 2 are presented in Table 10.

TABLE 10 OVERLAY THICKNESS USING AASHTO PROCEDURE

Project	E^{*}_{sg}	SN_{xeff}	SN_{y}	R_{LX}	R_{LY}	F_{RL}	T_{AC}
NDT Method 1							
	a) FWD						
King's Valley Highway	11,600	3.84	1.47	1.00	0.01	1.00	0.0
Willamina-Salem Highway	17,600	1.67	2.61	0.00	0.05	0.67	3.4
Lancaster Drive	8,600	3.96	2.90	1.00	0.07	1.00	0.0
Salem Parkway	22,700	5.34	2.58	1.00	0.05	1.00	0.0
Wilsonville-Hubbard Highway	6,000	6.00	3.35	1.00	0.10	1.00	0.0
	b) Dynaflect						
King's Valley Highway	35,100	3.17	0.87	0.46	0.00	0.63	0.0
Willamina-Salem Highway	53,700	1.75	1.70	0.00	0.01	0.71	1.0
Lancaster Drive	18,000	4.33	2.21	1.00	0.03	1.00	0.0
Salem Parkway	32,300	3.80	2.26	0.29	0.03	0.53	0.0
Wilsonville-Hubbard Highway	8,500	6.00	2.96	1.00	0.07	1.00	0.0
NDT Method 2							
	a) FWD						
King's Valley Highway	11,600	3.70	1.47	1.00	0.01	1.00	0.0
Willamina-Salem Highway	17,600	2.92	2.61	0.30	0.05	0.58	2.1
Lancaster Drive	8,600	3.99	2.90	1.00	0.07	1.00	0.0
Salem Parkway	26,300	6.95	2.45	1.00	0.04	1.00	0.0
Wilsonville-Hubbard Highway	7,300	6.00	3.12	1.00	0.08	1.00	0.0
	b) Dynaflect						
King's Valley Highway	35,100	8.15	0.87	1.00	0.00	1.00	0.0
Willamina-Salem Highway	53,700	7.26	1.70	1.00	0.01	1.00	0.0
Lancaster Drive	18,000	8.12	1.43	1.00	0.01	1.00	0.0
Salem Parkway	30,200	12.46	2.32	1.00	0.04	1.00	0.0
Wilsonville-Hubbard Highway	8,100	6.00	3.01	1.00	0.08	1.00	0.0

*Average Values

ODOT Method

The Oregon Department of Transportation (ODOT) employs the Caltrans deflection method with some modifications to design flexible overlays over flexible pavements (*1*). Deflection measurements are taken with the Dynaflect or FWD equipment, and the maximum deflection values are converted to equivalent Benkelman beam deflections for the Dynaflect and 9,000-lb load for the FWD. In the ODOT method, the highest 80th percentile deflection value is used in the evaluation in the following equation:

$$D_{80} = \overline{X} + 0.84\,S$$

where

D_{80} = design deflection value (80th percentile deflection);
$\overline{X}$ = mean deflection; and
S = standard deviation.

The representative deflection for a particular project length is then compared with a tolerable deflection that is a function of equivalent axle load repetition and thickness of the in-place pavement that has remaining fatigue life. For pavements that are substantially or wholly failed in fatigue, the tolerable deflection is based on the proposed overlay thickness only. An iterative procedure is then used to find the overlay thickness. If the representative deflection is less than the tolerable deflection, then an overlay is not needed. If the representative deflection is greater than the tolerable deflection, then the percent reduction in deflection is calculated as follows:

$$\text{percent reduction} = 100 * (D_{80} - D_t)/D_{80}$$

where D_t equals tolerable deflection.

The value of percent reduction is used to determine the gravel equivalency factor, which means that 1 in. thick of asphalt concrete is equivalent to certain inches thick of gravel

(*4*). The equivalent factor ranges from 1.52 to 2.5. A factor of 2.0 is used for this study.

A summary of the results for the ODOT method for the flexible pavement sites at King's Valley Highway, Willamina-Salem Highway, Lancaster Drive, and Salem Parkway is presented in Table 11.

DISCUSSION OF FINDINGS

The following discussion covers backcalculation procedures, NDT devices, determination of existing pavement structural capacity, and overlay design methods, since they are believed to be crucial to the implementation of the AASHTO Guide on overlay design.

Backcalculation Procedures

Backcalculation plays an important role in determining the strength of a pavement nondestructively. The result would influence the determination of the existing pavement structural capacity and, consequently, the overlay design. In this study, three backcalculation programs were used. With BISDEF and ELSDEF, a maximum of three iterations with a tolerance of 10 percent were specified. The modulus range and seed modulus were selected to be the same for each test location. With MODCOMP2, a maximum of twenty iterations with a tolerance of 0.15 percent were used. This tight tolerance range could be a reason for no solutions. The seed modulus used as an initial value to start the backcalculation was the same as those used in BISDEF and ELSDEF. The modulus range is not required for this program. Experience obtained from using these backcalculation programs shows that the predicted moduli may vary for each program. It is therefore necessary to use engineering judgment to ensure the calculated moduli are reasonable.

For conventional pavement structures, procedure 3 seems to work best and is recommended for backcalculation analysis. For other types of pavement structures, such as PCC and an AC surface with cement-treated base, laboratory tests on cores may be necessary, and the test result may be used to aid in determining other layers' moduli.

For a distressed PCC pavement, the NDT testing at slab center may not pick up problems at the joint. This is because of strength discontinuity between distressed slabs. For such cases, the NDT test should be performed at both the center and joint of the slab and the results evaluated separately.

NDT Devices

The backcalculated moduli, the existing pavement structural capacity (SN_{xeff}), and, consequently, the overlay thicknesses vary with the type of NDT device used. In this study, the deflection data from the Dynaflect result in a higher subgrade modulus. This is especially true for the NDT method 2. This may be because of the smaller deflection generated by the Dynaflect load and the stress sensitivity of subgrade material property. For NDT method 1, the value of the subgrade modulus has no effect on the determination of the existing pavement structural capacity (SN_{xeff}). However, in NDT method 2, this value can influence SN_{xeff}.

It is also noted that deflection values generated from FWD and Dynaflect are not linearly correlated. For instance, a 9,000-lb FWD load results in a deflection 20.76 mils at plate center (refer to Table 2, reading number 2 of King's Valley Highway), while a 1,000-lb Dynaflect load would have a 1.12-mil deformation at the same test point. The load ratio is 9, while the deflection ratio is 18, twice as high as the load ratio. Generally, for the three conventional types of pavement structure, the deflection ratio ranges from 16 to 22. For the cement-treated base/subbase project at Salem Parkway, the deflection ratio is about the same as the load ratio. For the PCC pavement at Wilsonville-Hubbard Highway, a deflection ratio ranging from 8 to 14 is identified. Because of these differences, it is difficult to tell which NDT device would provide a better indication of the pavement response. Based on the available data only, it is found that the deflection data from different NDT devices do have considerable influence on the backcalculated moduli, which in turn affect the resulting overlay design thickness.

The sensor space setting is an important factor to consider before taking deflection measurements. It is necessary to ensure that the last sensor be far enough away to obtain the pavement response purely from the subgrade. In this study, the last sensor location was 36 in. for FWD and 48 in. for Dynaflect. These configurations may not be appropriate for a good estimate of the subgrade modulus.

TABLE 11 OVERLAY THICKNESS USING ODOT PROCEDURE

	FWD			Dynaflect		
Project	D_{80}*	D_t	T_{AC} (in.)	D_{80}**	D_t	T_{AC} (in.)
King's Valley Highway	22	23	0	18	23	0
Willamina-Salem Highway	40	14	7.8	42	14	8.1
Lancaster Drive	28	16	1.0	28	16	1.0
Salem Parkway	6	19	0	5	19	0

*Deflection in mils.
**Converted to Benkelman beam value using the equation $BB = 15D^{1.3}$

Determination of SN_{xeff}

The existing structural capacity (SN_{xeff}) of a pavement can be determined using either NDT method 1 or NDT method 2, although the background of these two methods differs. For NDT method 1, the determination of SN_{xeff} relies on deflection basin data, methods of estimating the modulus of each pavement layer, and the relationship between layer modulus and layer coefficient. For NDT method 2, SN_{xeff} is determined from the maximum deflection as well as the in situ subgrade modulus. Ideally, these two methods should provide similar solutions. The results in this study (Table 9) show that the calculated SN_{xeff} using FWD data seems to give good correlation, while the SN_{xeff} values are less related when Dynaflect data are used. It should be noted that for NDT method 1, the layer coefficient for AC is restricted to a maximum value of 0.44; any modulus higher than 500 ksi would not contribute to the SN_{xeff} value. For NDT method 2, there is no such restriction; the SN_{xeff} is determined from the matching of the maximum deflection.

Overlay Design Methods

The overlay thicknesses determined from the two design methods, AASHTO and ODOT, are summarized in Table 12. As can be seen from the table, the King's Valley Highway and Salem Parkway projects have no need of an overlay. However, both procedures indicate that the Willamina-Salem Highway requires an overlay. The thickness of the required overlay varies for each method: the AASHTO NDT methods 1 and 2 require an overlay thickness ranging from 1 to 3.4 in., while an overlay thickness of about 8 in. is required by the ODOT method. For the Lancaster Drive site, no overlay is required using both NDT methods 1 and 2, while results from the ODOT method show that an overlay of 1 in. is required. Since Lancaster Drive had been overlaid the previous year (1986), it would seem that the AASHTO procedure provided a more reasonable estimate. The Wilsonville-Hubbard Highway is a PCC pavement. The structural capacity of this pavement is good, and no overlay is needed, as calculated using the AASHTO method. The pavement condition is bad, however, and cracking of slabs was found during the condition survey. It is possible that the determination of the SN_{xeff} was not right. The existence of the slab cracking seems difficult to identify using the AASHTO method.

Preliminary analysis of these results seems to lead to either one of the following conclusions: the ODOT method provides an overdesign of the overlay thickness and the AASHTO method(s) provide a more reasonable result; or the ODOT method provides a reasonable design and the AASHTO method(s) provide an underdesign. Further large-scale investigation on typical asphalt concrete and PCC pavements is needed to verify the conclusions reached thus far.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

Preliminary conclusions made from the data analyzed include:

1. The AASHTO method is based on the concept of reliability of design as well as the remaining life of the pavement; the ODOT method is based on the highest 80th percentile deflection value while the remaining life of the pavement is ignored.
2. A reliable backcalculation program is critical to implement NDT method 1. The study in this paper shows that the three backcalculation programs seem to work relatively well for the conventional pavement sections analyzed.
3. Good correlation was found between NDT methods 1 and 2 in determining the structural capacity using the FWD data. However, significant differences were also noted while using Dynaflect data.
4. The deflection data collected using FWD or Dynaflect can result in different overlay design. This conclusion is applied to both AASHTO and ODOT methods.

TABLE 12 COMPARISON OF TWO OVERLAY DESIGN METHODS

	AASHTO					
	NDT Method 1		NDT Method 2		ODOT	
Project Site	FWD	Dynaflect	FWD	Dynaflect	FWD	Dynaflect
King's Valley Highway*	0	0	0	0	0	0
Willamina-Salem Highway	3.4	1.0	2.1	0	7.8	8.1
Lancaster Drive	0	0	0	0	1.0	1.0
Salem Parkway	0	0	0	0	0	0
Wilsonville-Hubbard Highway**	0	0	0	0	N/A	

*This pavement would probably require a chip seal to prevent water infiltration. Its structural capacity is good.
**This is a PCC pavement. The structural capacity of this pavement is good; the thickness of the overlay would be controlled by reflection cracking.
N/A = not applicable.

Recommendations for Implementation

The following recommendations are based upon the results of this study:

1. Although the backcalculation program may produce a set of moduli for a pavement structure, laboratory tests may still be necessary for providing an estimate and/or verification of the backcalculated values. In some cases, the laboratory results should be used in the program as a fixed input to determine the moduli of the other layers. For conventional pavements, subgrade modulus may be estimated using the AASHTO equation and used as a fixed input to solve for other layers' moduli.
2. Engineering judgment must be made in selecting the layer coefficients. This is particularly difficult when extremely high and low moduli are involved.
3. A comparison of the two design procedures (AASHTO and ODOT) reveals significant differences between the calculated overlay designs. The reason for these differences needs to be understood before the 1986 AASHTO procedure can be applied to routine design work.

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REFERENCES

1. *Flexible Pavement Design Procedure.* Oregon State Highway Division, Salem, 1956.
2. *Thickness Design for Concrete Highway and Street Pavements.* Portland Cement Association, Skokie, Ill., 1984.
3. *AASHTO Interim Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C., 1981.
4. *Asphalt Concrete Overlay Design Manual.* California Department of Transportation, Sacramento, 1979.
5. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C., 1986.
6. H. Zhou, R. G. Hicks, and R. Noble. *Development of an Improved Overlay Design Procedure for Oregon: Vol. III—Field Manual.* FHWA-OR-RD-88-03C. FHWA, U.S. Department of Transportation, November 1987.
7. O. Tholen. *Falling Weight Deflectometer, A Device for Bearing Capacity Measurements: Properties and Performance.* Department of Highway Engineering, Royal Institute of Technology, Stockholm, Sweden, 1980.
8. O. Tholen, J. Sharma, and R. Terrel. Comparison of Falling Weight Deflectometer with Other Deflection Testing Devices. In *Transportation Research Record 1007,* TRB, National Research Council, Washington, D.C., 1985, pp. 20–26.
9. A. J. Bush and D. R. Alexander. Pavement Evaluation Using Deflection Basin Measurement and Layered Theory. In *Transportation Research Record 1022,* TRB, National Research Council, Washington, D.C., 1985, pp. 16–25.
10. A. J. Bush III. *Nondestructive Testing for Light Aircraft Pavements: Phase II—Development of Nondestructive Evaluation Methodology.* Final Report FAA-RD-80-9-II. FAA, U.S. Department of Transportation, November 1980.
11. R. L. Lytton, F. L. Roberts, and S. Stoffels. *Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing.* Final Report, NCHRP, Project 10-27. TRB, National Research Council, Washington, D.C., 1986.
12. R. G. Hicks and R. L. McHattie. *Use of Layered Theory in the Design and Evaluation of Pavement Systems.* Report FHWA-AK-RD-83-8. FHWA, U.S. Department of Transportation, July 1982.
13. L. H. Irwin. *User's Guide to MODCOMP2, Version 2.1.* Local Roads Program, Cornell University, Ithaca, N.Y., November 1983.

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