# DEVELOPMENT OF AN IMPROVED OVERLAY DESIGN PROCEDURE FOR OREGON

## VOLUME II - EVALUATION OF PROCEDURE

by

Haiping Zhou
Research Assistant
Oregon State University

R.G. Hicks
Professor of Civil Engineering
Oregon State University

T. Rwebangira
Research Assistant
Oregon State University

and

Ronald Noble
Materials Unit Engineer
Oregon State Highway Division

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16. Abstract

This report is the second in a three-volume series dealing with the development of an improved overlay design procedure for Oregon. This report presents the results of the second year findings. Data from five projects were collected and analyzed using both NDT methods 1 and 2 from the 1986 AASHTO Guides. The overlay thickness using the AASHTO procedure were compared with those using the Caltrans and Oregon DOT methods.

Though the results indicate there is reasonable comparisons between the various methods, the authors have concluded that:

1) NDT method 1 still needs further work, particularly in developing reliable backcalculation methods.

2) NDT method 2 can be used now with reasonable confidence.

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## DISCLAIMER

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# 1.0 INTRODUCTION

## 1.1 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 throughout the state (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). Presently, the Dynaflect and 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 effective thickness of the existing pavement (PCA and AASHTO methods) from the new design thickness.

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 the designers to make better evaluations of 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 remaining life.

In Volume I of this report, a framework for a new design procedure was presented. In essence, the report recommended the implementation of the 1986 AASHTO Guide procedure (5) for overlay design.

## 1.2 Purpose

The purpose of this report 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 material sampling,
2) Laboratory testing of materials sampled from each project,
3) Analysis of deflection basin data and development of overlay design recommendations,
4) Discussion of results, and
5) Development of appropriate conclusions and recommendations.

## 2.0 1986 AASHTO OVERLAY DESIGN METHOD

The AASHTO method can best be summarized by presenting each of its components separately. Seven steps are used to outline the inputs for the AASHTO procedure.

The first step identifies the homogeneous sections of the highway to be tested for deflection. This is a function of the type and extent of pavement distress and the amount of available historic data for that particular highway. This step is routinely performed for every overlay procedure and there should be no difficulty in developing these sections.

The second step evaluates the cumulative traffic prior to the overlay and determines the future expected traffic. Traffic is an important consideration in all overlay designs. Accurate estimates of traffic must be made for the procedure to produce a valid or realistic overlay thickness. Future traffic expectations should be no problem, since estimates of this type are commonly made for all overlay design procedures. However, estimates of previous traffic may be difficult to obtain, especially for older,low volume roads. Prior traffic data is not required if the NDT approach is used to determine remaining life.

The third step determines the material characteristics for each pavement layer and requires the most effort. This step is one which will be unfamiliar to users since most overlay design procedures do not consider the properties of the in situ pavement layer material. The subgrade and pavement layer properties must be reliably determined to ascertain the structural strength and the remaining life of the pavement. These properties can be calculated if the variables of the NDT equipment and the associated deflection values for the applied load are known (6,7,8). The moduli values can be backcalculated using computer programs such as BISDEF, ELSDEF, or MODCOMP2 (9,10,11). These programs approximate the layer moduli from the obtained deflection values and the known load applied to the pavement structure. There are some assumptions and limitations for these programs which may affect the degree of reliability obtained from the calculated layer moduli values (12). However, if the range for the material is well bracketed, the programs will provide much closer estimations of the pavement layer moduli. This may involve taking cores and performing laboratory tests to obtain an accurate estimate of moduli value for the surface layer.

The <u>fourth step</u> determines the effective structural capacity ($SC_{xeff}$) of the existing pavement. This is dependent upon the type of structure to be overlaid. For existing portland cement concrete (PCC) pavements, the effective structural capacity ($D_{xeff}$) can be determined using NDT method 1 or other approximate procedures. With NDT method 1, $D_{xeff}$ is determined from the concrete layer modulus and the thickness of the PCC layer. The thickness may be determined from construction records or coring. The modulus is obtained from backcalculation or from tests on cores. With non-NDT approximate procedures, three alternative approaches may be used: visual condition factor, nominal size of PCC slab fragments, and/or remaining life. These three approaches are somewhat equivalent in determining the $D_{xeff}$. Backcalculation is not required in these procedures.

For flexible pavements, the effective structural capacity ($SC_{xeff}$) can be determined using either NDT method 1 or NDT method 2. With NDT method 1, $SC_{xeff}$ is a function of the layer moduli determined from Step 3 for each layer. Layer coefficients are assigned to each layer according to their relative strength. The layer thicknesses are determined from construction records or from coring. The sum of the product of layer coefficients and the thickness for each layer yields the structural number of the pavement. With NDT method 2, $SC_{xeff}$ can be estimated from the in situ subgrade modulus and the maximum measured pavement deflection, provided the characteristics of the particular NDT dynamic device are known.

The <u>fifth step</u> determines the future structural capacity ($SC_y$) of the pavement and is the equivalent of a new structural design. The future structural capacity is determined either by equations or through the use of nomographs. The equations require traffic from Step 2, reliability and standard deviation, the subgrade modulus obtained in Step 3, and desired serviceability loss levels. The reliability factor and standard deviation (level of confidence that a pavement will not fail within a specified period) are selected by the engineer using the 1986 AASHTO Guide (5). The last input value needed is the loss in the present serviceability index value from a new pavement to an unacceptable pavement.

The <u>sixth step</u> in the overlay design process calculates the remaining life factor ($F_{RL}$). Several methods are presented for the determination of the remaining life of the in situ pavement and the future overlaid pavement. The AASHTO Guide recommends the use of the NDT approach for fatigued pavements and the traffic approach for newer pavements. It may be difficult to determine the cumulative traffic that a highway has experienced unless adequate records have been maintained. If more precise historical traffic volumes could be obtained, a more appropriate and economical overlay can be recommended. There may be a significant difference in $F_{RL}$ values depending upon which method is chosen for determination.

The <u>seventh and last step</u> substitutes the calculated values ($SC_{xeff}$, $SC_y$, and $F_{RL}$) into the appropriate equation to determine the structural number for overlay ($SC_{OL}$). If the $SC_{OL}$ value is greater than zero, an overlay is needed. For flexible types of overlay, the required thickness is found by dividing $SC_{OL}$ ($SN_{OL}$) by the layer coefficient of the surface material being overlaid. For rigid types of overlay, the thickness can be determined by subtracting remaining effective structural capacity from a newly required pavement thickness.

# 3.0 FIELD DATA COLLECTION

This section of the report describes the projects evaluated and presents the results of the deflection measurements.

## 3.1 Project Descriptions

Field data were collected in the spring of 1987 on five project sites on existing highways in the state of Oregon. Four of the project sites were flexible pavements while the fifth site was a rigid pavement. The age of projects ranged from 10 to 25 years. Figure 1 shows the location of the project sites and Figure 2 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 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.

To obtain an estimate of existing asphalt concrete layer material, cores were taken on each project site. These cores were tested in the laboratory to get an estimate of the resilient modulus of the surfacing layer. The results are presented in Chapter 4.0.

The existing pavement conditions for the five sites varied considerably from one to the other. Three of the test sites (King's Valley Highway, Salem Parkway, and Lancaster Drive) did not show any signs of pavement surface distress. The Lancaster Drive site had been overlaid the previous year and, at the time of testing its surface, was in an excellent condition. The Salem

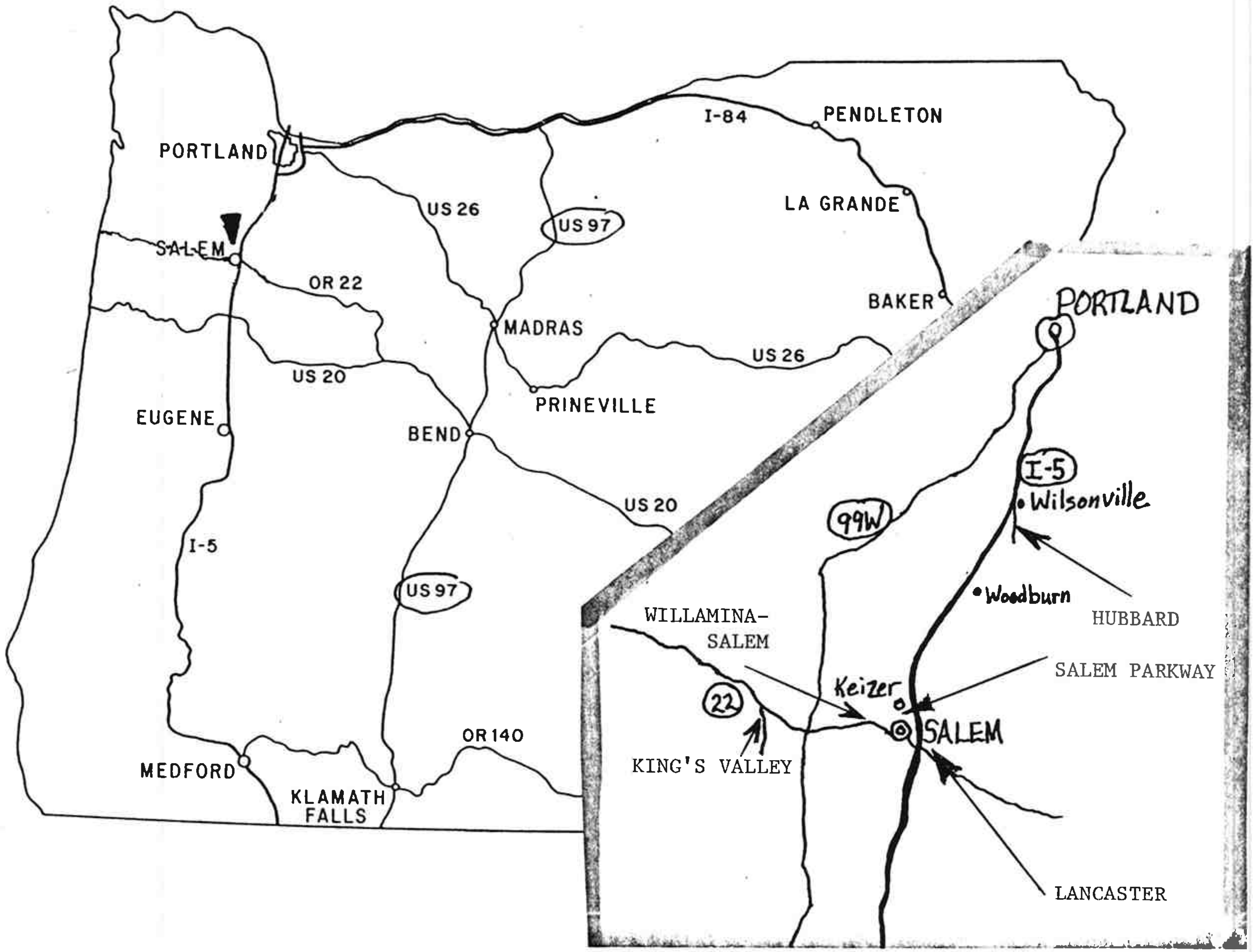



Figure 1. Location Map of the Project Sites.

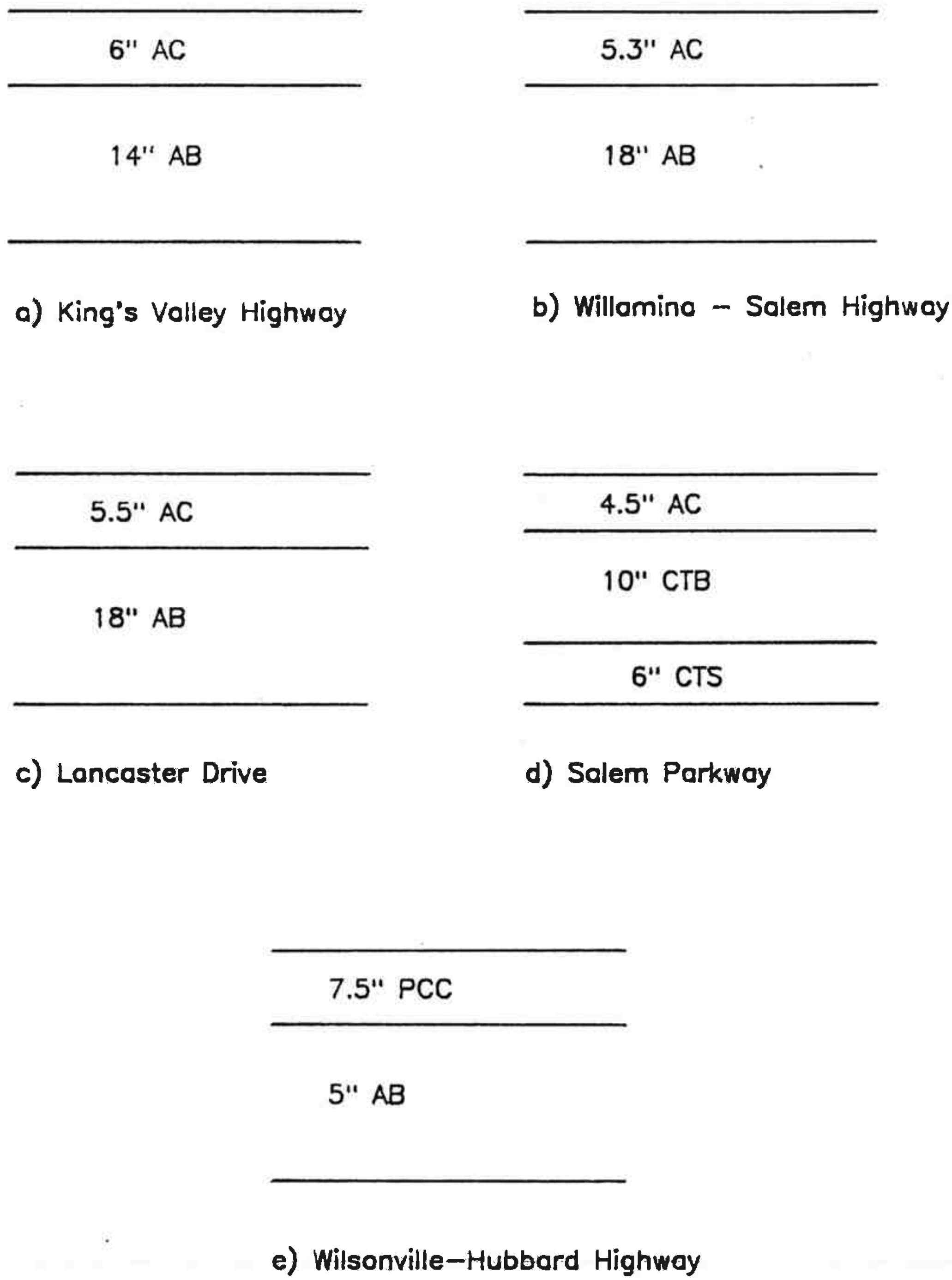



Figure 2. Cross Sections of Pavements Analyzed.

Table 1. Summary of Project Data.

| Project | X-Section | Traffic | Pavement Condition |
|---|---|---|---|
| King's Valley Highway | 6.0" AC<br>14.0" Agg. Base<br>Subgrade R = 7 | 4500 ESAL/yr<br>10 yr TC = 6.0<br>Cumulative ESAL = $5 \times 10^4$<br>Future traffic = 33,200 | Good surface condition<br>Drainage adequate |
| Willamina-Salem Highway | 5.3" AC<br>18.0" Agg. Base<br>Subgrade R = 5 | Current = 173,200 ESAL/yr<br>10 yr TC = 9.7<br>Cumulative ESAL = $2 \times 10^6$<br>Future traffic = 1,876,600 | Fair - Poor<br>Longitudinal and alligator cracking in all lanes<br>Evidence of rutting on outside lanes |
| Lancaster Drive | 5.5" AC<br>18.0" Agg. Base<br>Subgrade R = 6 | Total Accum. 15 years<br>~40,000 ESAL/yr<br>20 yr Future traffic = 1,000,000 | Surface condition very good<br>Drainage good |
| Salem Parkway | 4.5" AC<br>10.0" CTB<br>6.0" CTS<br>Subgrade R = 5 | Current = 135,000 ESAL/yr<br>Accumulative = 420,000 ESAL<br>20 yr Future traffic = 3,200,000 | Surface condition and drainage very good |
| Wilsonville-Hubbard Highway | 7.5" PCC<br>5.0" Agg. Base<br>Subgrade R = 5 | Current = 115,000 ESAL/yr<br>10 yr TC = 9.1<br>Future traffic = 1,097,300 | Good - Poor<br>Cracking of slab<br>Erosion of shoulders |

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

Parkway and King's Valley Highway sites did not show any signs of distress either. 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. Photos of the pavement condition as of April 1987 are given in Appendix A.

## 3.2 Pavement Deflection Measurements

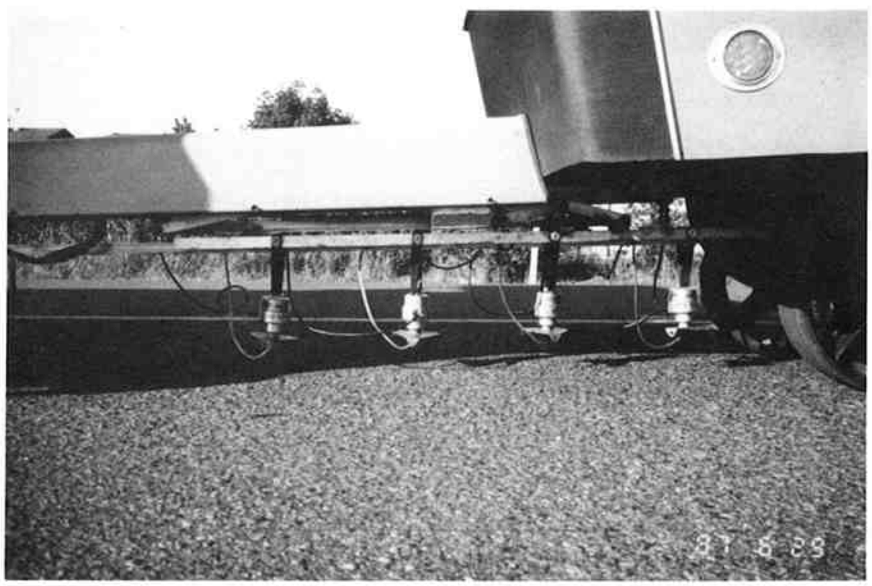
Pavement surface deflection data were measured at 50-ft intervals for 1000-ft sections for each project. The measurements were taken with the KUAB Falling Weight Deflectometer (FWD) and the Dynaflect, both owned and operated by ODOT. For each site, deflection basin measurements were taken at 50-ft intervals in the outer wheel path. The FWD data were taken at three load levels and converted to a 9000-lb load level by simple linear interpolation. The Dynaflect data were measured at 1000-lb cyclic load and at a frequency of 8 Hz. Both the Dynaflect and FWD tests were conducted at the same locations so direct comparisons could be made.

### 3.2.1 Deflection Equipment

The Dynaflect, owned by ODOT, employs two counter-rotating masses to apply a peak-to-peak dynamic force of 1000 lbs (4.4 kN) at a fixed frequency of 8 Hz (see Figure 3). 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 KUAB Falling Weight Deflectometer, owned by ODOT, was also used to measure surface deflection (Figure 4). This device is trailer-mounted and towed by a 3/4-ton van. The impulse force is created by dropping a set of two

a) Overview



b) Closeup of Sensors

Figure 3. Photo of Oregon DOT's Dynaflect.


a) Overview


b) Closeup of Internal Working System

Figure 4. Photo of Oregon DOT's KUAB Falling Weight Deflectometer.

weights from different heights. By varying the drop height, the load at the pavement surface was varied from 4900 to 11,300 lbs. The two-mass system is used to create a smooth load pulse similar to that created by a moving wheel load (6,7). 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 of magnitude equal to that produced by a loaded truck, there is no need to correct the determined in situ moduli for stress sensitivity. The load configuration for both FWD and Dynaflect is shown in Figure 5.

### 3.2.2 Deflection Results

Tables 2 to 6 show deflection values from the five sites tested. For each site the deflection readings were recorded by the two NDT test devices, with a 9000-lb load level for the FWD and a 1000-lb load level for the Dynaflect. The deflection value at the 9000-lb load level was obtained by linear interpolation and is used as the input in the backcalculation procedures. The value of 9000 lbs was based on the wheel load of a standard axle of 18000 lbs commonly used in the United States. Each table shows deflection values for the various sensor positions for both the FWD and Dynaflect. Figures 6 to 10 show the deflection measurements using both FWD and Dynaflect along the test section on five projects. Deflection values measured at each sensor location are averaged and plotted as shown in Figures 11 and 12. These results provide an illustration on the variation of deflection measurement along the road section and on each project site. It can be seen immediately from Figures 11 and 12 that Willamina-Salem Highway has highest deflection at NDT device load center. This may infer that this particular roadway has the lowest structural capacity among the five projects.

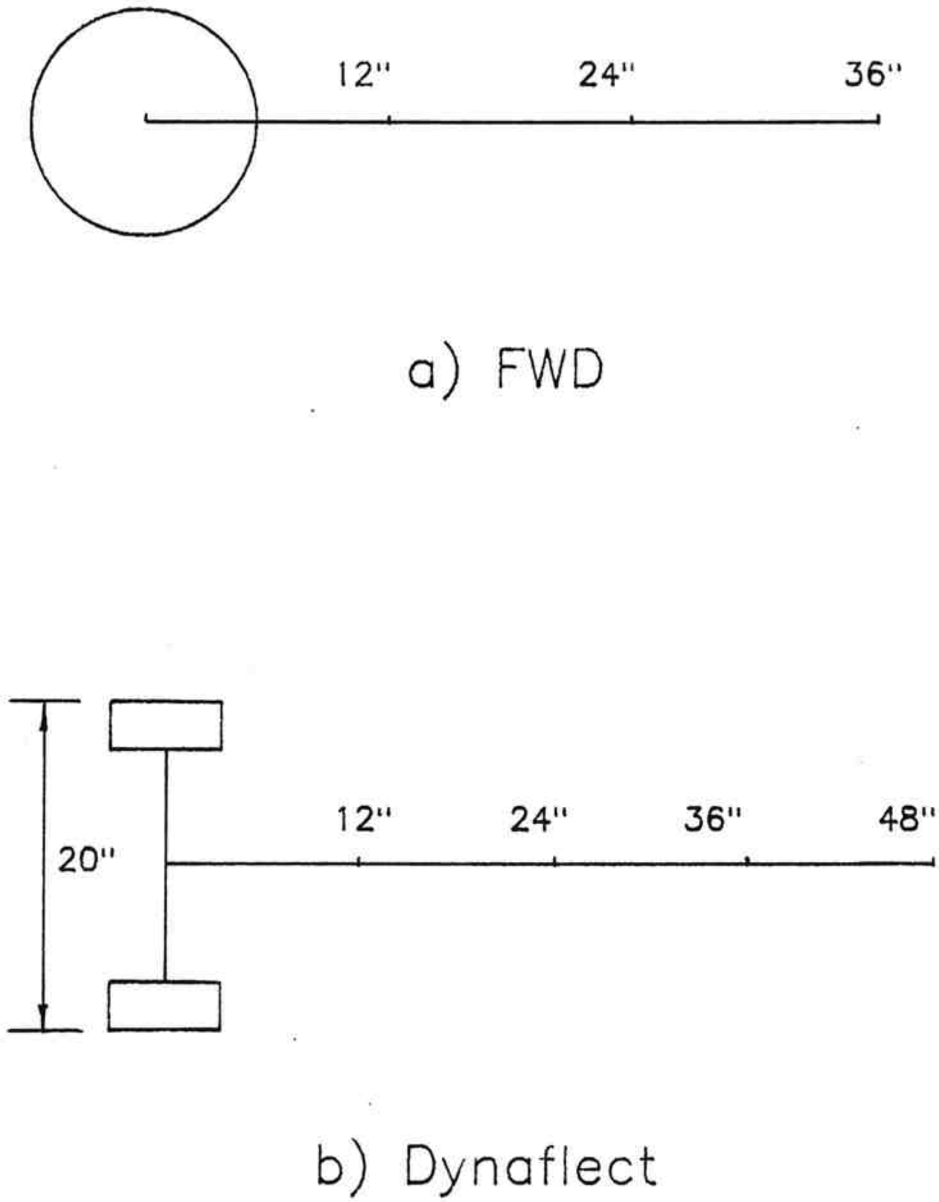



Figure 5. Load Configuration for both NDT Test Units.

Table 2. Deflection Values for King's Valley Highway (Temp = 60°F).

| Reading Number | Equipment | Load (lbs) | Sensors ($x10^{-3}$) in. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| 1 | FWD | 9000 | 19.5 | 14.9 | 9.0 | 4.8 | | |
| | Dynaflect | 1000 | 1.06 | 0.76 | 0.43 | 0.25 | 0.15 | |
| 2 | FWD | 9000 | 16.9 | 13.1 | 8.2 | 4.8 | | |
| | Dynaflect | 1000 | 0.97 | 0.72 | 0.41 | 0.23 | 0.14 | |
| 3 | FWD | 9000 | 20.9 | 16.1 | 10.6 | 6.31 | | |
| | Dynaflect | 1000 | 1.02 | 0.76 | 0.47 | 0.27 | 0.16 | |
| 4 | FWD | 9000 | 20.76 | 16.14 | 10.67 | 6.25 | | |
| | Dynaflect | 1000 | 1.12 | 0.81 | 0.49 | 0.28 | 0.16 | |
| 5 | FWD | 9000 | 22.17 | 16.79 | 10.52 | 5.72 | | |
| | Dynaflect | 1000 | 1.26 | 0.90 | 0.51 | 0.28 | 0.17 | |
| 6 | FWD | 9000 | 22.66 | 18.35 | 10.38 | 5.73 | | |
| | Dynaflect | 1000 | 1.04 | 0.75 | 0.47 | 0.28 | 0.17 | |
| 7 | FWD | 9000 | 18.06 | 14.40 | 9.55 | 5.94 | | |
| | Dynaflect | 1000 | 1.18 | 0.82 | 0.47 | 0.27 | 0.15 | |
| 8 | FWD | 9000 | 22.19 | 16.97 | 10.47 | 5.77 | | |
| | Dynaflect | 1000 | 0.95 | 0.74 | 0.47 | 0.29 | 0.17 | |
| 9 | FWD | 9000 | 17.90 | 14.00 | 10.01 | 5.82 | | |
| | Dynaflect | 1000 | 1.17 | 0.86 | 0.51 | 0.30 | 0.18 | |
| 10 | FWD | 9000 | 18.24 | 11.74 | 9.57 | 5.46 | | |
| | Dynaflect | 1000 | 0.93 | 0.72 | 0.46 | 0.28 | 0.16 | |
| 11 | FWD | 9000 | 15.47 | 12.08 | 8.52 | 5.09 | | |
| | Dynaflect | 1000 | 0.96 | 0.72 | 0.46 | 0.28 | 0.17 | |
| 12 | FWD | 9000 | 15.63 | 12.30 | 8.53 | 5.13 | | |
| | Dynaflect | 1000 | 1.00 | 0.71 | 0.42 | 0.23 | 0.13 | |
| 13 | FWD | 9000 | 17.87 | 13.27 | 8.04 | 4.43 | | |
| | Dynaflect | 1000 | 1.29 | 0.88 | 0.49 | 0.27 | 0.16 | |
| 14 | FWD | 9000 | 22.68 | 16.55 | 10.07 | 5.36 | | |
| | Dynaflect | 1000 | 1.29 | 0.85 | 0.42 | 0.20 | 0.11 | |
| 15 | FWD | 9000 | 23.95 | 17.55 | 9.85 | 5.00 | | |
| | Dynaflect | 1000 | 0.99 | 0.70 | 0.39 | 0.20 | 0.11 | |
| 16 | FWD | 9000 | 18.65 | 14.23 | 9.83 | 4.58 | | |
| | Dynaflect | 1000 | 1.02 | 0.73 | 0.40 | 0.21 | 0.11 | |
| 17 | FWD | 9000 | 18.12 | 13.23 | 8.34 | 4.53 | | |
| | Dynaflect | 1000 | 0.98 | 0.69 | 0.38 | 0.21 | 0.12 | |
| 18 | FWD | 9000 | 22.27 | 16.39 | 9.88 | 5.02 | | |
| | Dynaflect | 1000 | 1.09 | 0.77 | 0.43 | 0.24 | 0.14 | |
| 19 | FWD | 9000 | 21.43 | 15.74 | 9.99 | 5.43 | | |
| | Dynaflect | 1000 | 1.00 | 0.75 | 0.44 | 0.24 | 0.14 | |

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Table 3. Deflection Values for Willamina-Salem Highway (Temp = 68°F).

| Reading Number | Equipment | Load (lbs) | Sensors ($x10^{-3}$) in. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| 1 | FWD | 9000 | 31.73 | 21.08 | 10.7 | 4.96 | | |
| | Dynaflect | 1000 | 1.84 | 1.08 | 0.55 | 0.26 | 0.13 | |
| 2 | FWD | 9000 | 26.34 | 18.63 | 10.14 | 4.73 | | |
| | Dynaflect | 1000 | 2.04 | 1.19 | 0.57 | 0.26 | 0.13 | |
| 3 | FWD | 9000 | 31.32 | 19.85 | 9.55 | 4.15 | | |
| | Dynaflect | 1000 | 1.83 | 1.04 | 0.46 | 0.21 | 0.12 | |
| 4 | FWD | 9000 | 36.57 | 22.65 | 10.39 | 4.47 | | |
| | Dynaflect | 1000 | 1.76 | 0.98 | 0.42 | 0.20 | 0.12 | |
| 5 | FWD | 9000 | 32.93 | 19.74 | 7.23 | 2.90 | | |
| | Dynaflect | 1000 | 1.93 | 1.00 | 0.38 | 0.16 | 0.09 | |
| 6 | FWD | 9000 | 37.18 | 22.97 | 9.14 | 2.92 | | |
| | Dynaflect | 1000 | 2.08 | 1.08 | 0.41 | 0.17 | 0.10 | |
| 7 | FWD | 9000 | 42.35 | 25.12 | 10.62 | 3.98 | | |
| | Dynaflect | 1000 | 2.27 | 1.17 | 0.45 | 0.20 | 0.10 | |
| 8 | FWD | 9000 | 43.82 | 27.36 | 11.18 | 3.69 | | |
| | Dynaflect | 1000 | 2.39 | 1.09 | 0.37 | 0.16 | 0.10 | |
| 9 | FWD | 9000 | 37.77 | 23.12 | 7.34 | 2.41 | | |
| | Dynaflect | 1000 | 2.32 | 1.17 | 0.36 | 0.16 | 0.10 | |
| 10 | FWD | 9000 | 40.15 | 24.47 | 6.96 | 0.90 | | |
| | Dynaflect | 1000 | 1.91 | 0.97 | 0.27 | 0.12 | 0.07 | |
| 11 | FWD | 9000 | 36.78 | 21.94 | 8.61 | 2.23 | | |
| | Dynaflect | 1000 | 1.79 | 0.96 | 0.38 | 0.13 | 0.07 | |
| 12 | FWD | 9000 | 36.77 | 22.22 | 9.50 | 3.32 | | |
| | Dynaflect | 1000 | 1.84 | 1.03 | 0.43 | 0.17 | 0.08 | |
| 13 | FWD | 9000 | 28.77 | 19.14 | 9.36 | 4.18 | | |
| | Dynaflect | 1000 | 1.87 | 1.09 | 0.50 | 0.25 | 0.15 | |
| 14 | FWD | 9000 | 29.70 | 17.72 | 8.98 | 4.31 | | |
| | Dynaflect | 1000 | 1.74 | 1.01 | 0.49 | 0.24 | 0.13 | |
| 15 | FWD | 9000 | 35.50 | 22.45 | 9.36 | 3.18 | | |
| | Dynaflect | 1000 | 1.99 | 1.11 | 0.48 | 0.21 | 0.12 | |
| 16 | FWD | 9000 | 39.80 | 24.70 | 8.95 | 3.76 | | |
| | Dynaflect | 1000 | 1.90 | 1.07 | 0.47 | 0.23 | 0.14 | |
| 17 | FWD | 9000 | 44.93 | 26.38 | 11.43 | 4.74 | | |
| | Dynaflect | 1000 | 2.03 | 1.09 | 0.52 | 0.28 | 0.15 | |
| 18 | FWD | 9000 | 33.33 | 18.18 | 7.13 | 2.81 | | |
| | Dynaflect | 1000 | 2.37 | 1.20 | 0.52 | 0.31 | 0.23 | |
| 19 | FWD | 9000 | 33.75 | 20.37 | 8.19 | 3.66 | | |
| | Dynaflect | 1000 | 2.30 | 1.20 | 0.55 | 0.33 | 0.26 | |
| 20 | FWD | 9000 | 45.88 | 28.71 | 12.50 | 6.30 | | |
| | Dynaflect | 1000 | 2.38 | 1.22 | 0.49 | 0.29 | 0.21 | |
| 21 | FWD | 9000 | 27.48 | 17.80 | 8.22 | 2.66 | | |
| | Dynaflect | 1000 | 1.73 | 1.00 | 0.43 | 0.21 | 0.15 | |

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Table 4. Deflection Values for Lancaster Drive (Temp = 57°F).

| Reading Number | Equipment | Load (lbs) | Sensors ($x10^{-3}$) in. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| 2 | FWD | 9000 | 26.17 | 19.80 | 11.10 | 7.70 | | |
| | Dynaflect | 1000 | 1.55 | 1.11 | 0.71 | 0.46 | 0.31 | |
| 3 | FWD | 9000 | 27.30 | 20.30 | 12.08 | 7.23 | | |
| | Dynaflect | 1000 | 1.65 | 1.16 | 0.72 | 0.44 | 0.31 | |
| 4 | FWD | 9000 | 29.46 | 21.11 | 12.71 | 7.6 | | |
| | Dynaflect | 1000 | 1.62 | 1.26 | 0.80 | 0.49 | 0.33 | |
| 5 | FWD | 9000 | 24.50 | 17.67 | 10.27 | 6.78 | | |
| | Dynaflect | 1000 | 1.44 | 1.09 | 0.71 | 0.47 | 0.32 | |
| 6 | FWD | 9000 | 23.97 | 17.30 | 10.1 | 6.55 | | |
| | Dynaflect | 1000 | 1.50 | 1.14 | 0.72 | 0.46 | 0.32 | |
| 7 | FWD | 9000 | 23.56 | 16.97 | 9.81 | 6.32 | | |
| | Dynaflect | 1000 | 1.70 | 1.20 | 0.75 | 0.43 | 0.32 | |
| 8 | FWD | 9000 | 32.49 | 23.53 | 14.91 | 8.82 | | |
| | Dynaflect | 1000 | 1.18 | 0.91 | 0.66 | 0.47 | 0.35 | |
| 9 | FWD | 9000 | 14.43 | 11.68 | 8.17 | 5.79 | | |
| | Dynaflect | 1000 | 1.04 | 0.88 | 0.68 | 0.50 | 0.36 | |
| 10 | FWD | 9000 | 13.56 | 11.00 | 7.73 | 5.48 | | |
| | Dynaflect | 1000 | 1.67 | 1.19 | 0.70 | 0.44 | 0.31 | |
| 11 | FWD | 9000 | 23.93 | 17.81 | 10.64 | 6.86 | | |
| | Dynaflect | 1000 | 1.62 | 1.18 | 0.76 | 0.49 | 0.34 | |
| 12 | FWD | 9000 | 24.12 | 17.96 | 11.20 | 7.39 | | |
| | Dynaflect | 1000 | 1.42 | 1.08 | 0.73 | 0.48 | 0.33 | |
| 13 | FWD | 9000 | 20.16 | 15.53 | 9.83 | 6.69 | | |
| | Dynaflect | 1000 | 1.65 | 1.20 | 0.78 | 0.49 | 0.34 | |
| 14 | FWD | 9000 | 26.39 | 21.13 | 12.62 | 7.94 | | |
| | Dynaflect | 1000 | 1.65 | 1.22 | 0.78 | 0.47 | 0.33 | |
| 15 | FWD | 9000 | 25.82 | 20.69 | 12.43 | 7.43 | | |
| | Dynaflect | 1000 | 1.38 | 1.10 | 0.76 | 0.51 | 0.35 | |
| 16 | FWD | 9000 | 23.47 | 18.73 | 11.38 | 7.12 | | |
| | Dynaflect | 1000 | 1.40 | 1.07 | 0.73 | 0.49 | 0.33 | |
| 17 | FWD | 9000 | 26.06 | 19.86 | 12.08 | 7.59 | | |
| | Dynaflect | 1000 | 1.36 | 0.97 | 0.62 | 0.39 | 0.26 | |
| 18 | FWD | 9000 | 23.98 | 18.39 | 11.96 | 7.59 | | |
| | Dynaflect | 1000 | 1.53 | 1.13 | 0.71 | 0.45 | 0.31 | |
| 19 | FWD | 9000 | 27.12 | 20.72 | 13.36 | 8.75 | | |
| | Dynaflect | 1000 | 1.44 | 1.11 | 0.76 | 0.51 | 0.37 | |
| 20 | FWD | 9000 | 21.67 | 17.56 | 11.64 | 8.00 | | |
| | Dynaflect | 1000 | 1.53 | 1.12 | 0.71 | 0.47 | 0.34 | |

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Table 5. Deflection Values for Salem Parkway (Temp = 67°F).

| Reading Number | Equipment | Load (lbs) | Sensors ($x10^{-3}$) in. 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|---|---|
| 2 | FWD | 9000 | 4.8 | 4.04 | 2.46 | 2.19 | | |
| | Dynaflect | 1000 | 0.40 | 0.31 | 0.27 | 0.23 | 0.19 | |
| 3 | FWD | 9000 | 6.67 | 4.62 | 3.82 | 3.40 | | |
| | Dynaflect | 1000 | 0.40 | 0.27 | 0.22 | 0.19 | 0.17 | |
| 4 | FWD | 9000 | 4.22 | 3.49 | 3.05 | 2.62 | | |
| | Dynaflect | 1000 | 0.35 | 0.29 | 0.25 | 0.21 | 0.18 | |
| 5 | FWD | 9000 | 6.46 | 5.15 | 4.14 | 3.73 | | |
| | Dynaflect | 1000 | 0.47 | 0.37 | 0.31 | 0.25 | 0.20 | |
| 6 | FWD | 9000 | 6.00 | 4.31 | 3.43 | 3.19 | | |
| | Dynaflect | 1000 | 0.42 | 0.38 | 0.31 | 0.25 | 0.20 | |
| 7 | FWD | 9000 | 5.22 | 4.47 | 3.26 | 3.11 | | |
| | Dynaflect | 1000 | 0.41 | 0.31 | 0.26 | 0.21 | 0.17 | |
| 8 | FWD | 9000 | 4.47 | 3.36 | 2.81 | 2.4 | | |
| | Dynaflect | 1000 | 0.35 | 0.29 | 0.26 | 0.21 | 0.15 | |
| 9 | FWD | 9000 | 4.89 | 3.49 | 2.48 | 2.06 | | |
| | Dynaflect | 1000 | 0.43 | 0.32 | 0.26 | 0.21 | 0.17 | |
| 10 | FWD | 9000 | 3.54 | 3.45 | 2.61 | 2.08 | | |
| | Dynaflect | 1000 | 0.37 | 0.30 | 0.24 | 0.19 | 0.15 | |
| 11 | FWD | 9000 | 3.94 | 2.65 | 2.33 | 1.87 | | |
| | Dynaflect | 1000 | 0.31 | 0.24 | 0.20 | 0.17 | 0.14 | |
| 12 | FWD | 9000 | 5.04 | 4.09 | 3.11 | 2.65 | | |
| | Dynaflect | 1000 | 0.39 | 0.32 | 0.26 | 0.21 | 0.17 | |
| 13 | FWD | 9000 | 5.18 | 4.38 | 3.57 | 3.17 | | |
| | Dynaflect | 1000 | 0.39 | 0.33 | 0.28 | 0.23 | 0.19 | |
| 14 | FWD | 9000 | 3.21 | 2.69 | 2.16 | 2.18 | | |
| | Dynaflect | 1000 | 0.29 | 0.27 | 0.24 | 0.22 | 0.19 | |
| 15 | FWD | 9000 | 5.78 | 3.99 | 3.42 | 2.83 | | |
| | Dynaflect | 1000 | 0.46 | 0.37 | 0.30 | 0.23 | 0.19 | |
| 16 | FWD | 9000 | 3.84 | 3.06 | 2.39 | 2.22 | | |
| | Dynaflect | 1000 | 0.36 | 0.32 | 0.28 | 0.24 | 0.21 | |
| 17 | FWD | 9000 | 4.90 | 3.77 | 2.90 | 2.27 | | |
| | Dynaflect | 1000 | 0.47 | 0.39 | 0.32 | 0.27 | 0.23 | |
| 18 | FWD | 9000 | 3.54 | 3.05 | 2.25 | 2.21 | | |
| | Dynaflect | 1000 | 0.35 | 0.30 | 0.26 | 0.23 | 0.20 | |
| 19 | FWD | 9000 | 4.95 | 4.31 | 2.84 | 2.21 | | |
| | Dynaflect | 1000 | 0.53 | 0.36 | 0.27 | 0.23 | 0.19 | |
| 20 | FWD | 9000 | 3.72 | 3.07 | 2.47 | 2.15 | | |
| | Dynaflect | 1000 | 0.48 | 0.40 | 0.37 | 0.33 | 0.30 | |
| 21 | FWD | 9000 | 3.82 | 3.27 | 2.76 | 2.53 | | |
| | Dynaflect | 1000 | 0.49 | 0.42 | 0.39 | 0.36 | 0.33 | |

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Table 6. Deflection Values for Wilsonville-Hubbard Highway.

| Reading Number | Equipment | Load (lbs) | Sensors ($x10^{-3}$) in. | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | 1 | 2 | 3 | 4 | 5 | 6 |
| 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 | 18.20 | 16.19 | 13.58 | 11.99 | | |
| | Dynaflect | 1000 | 1.01 | 0.96 | 0.86 | 0.76 | 0.67 | |
| 4 | FWD | 9000 | 11.93 | 11.16 | 8.73 | 7.64 | | |
| | Dynaflect | 1000 | 1.06 | 1.03 | 0.91 | 0.79 | 0.68 | |
| 5 | FWD | 9000 | 13.61 | 12.09 | 10.08 | 8.67 | | |
| | Dynaflect | 1000 | 0.99 | 0.93 | 0.82 | 0.69 | 0.56 | |
| 6 | FWD | 9000 | 13.24 | 11.06 | 8.56 | 7.20 | | |
| | Dynaflect | 1000 | 1.20 | 1.14 | 1.02 | 0.88 | 0.72 | |
| 7 | FWD | 9000 | 15.69 | 13.74 | 11.25 | 10.19 | | |
| | Dynaflect | 1000 | 1.09 | 1.06 | 0.97 | 0.85 | 0.72 | |
| 8 | FWD | 9000 | 12.63 | 10.26 | 7.71 | 6.34 | | |
| | Dynaflect | 1000 | 1.28 | 1.22 | 1.11 | 0.98 | 0.84 | |
| 9 | FWD | 9000 | 13.14 | 12.84 | 11.49 | 10.67 | | |
| | Dynaflect | 1000 | 1.41 | 1.28 | 1.11 | 0.92 | 0.74 | |
| 10 | FWD | 9000 | 13.22 | 11.84 | 9.71 | 8.30 | | |
| | Dynaflect | 1000 | 1.16 | 1.10 | 0.99 | 0.86 | 0.72 | |
| 11 | FWD | 9000 | 11.01 | 10.29 | 8.85 | 8.08 | | |
| | Dynaflect | 1000 | 1.20 | 1.12 | 1.00 | 0.87 | 0.74 | |
| 12 | FWD | 9000 | 12.17 | 9.95 | 7.54 | 6.39 | | |
| | Dynaflect | 1000 | 1.28 | 1.25 | 1.14 | 1.01 | 0.88 | |
| 13 | FWD | 9000 | 15.43 | 12.62 | 9.64 | 8.00 | | |
| | Dynaflect | 1000 | 1.15 | 1.12 | 1.02 | 0.89 | 0.75 | |
| 14 | FWD | 9000 | 17.49 | 15.46 | 12.43 | 10.50 | | |
| | Dynaflect | 1000 | 1.29 | 1.20 | 1.05 | 0.90 | 0.74 | |
| 15 | FWD | 9000 | 13.32 | 10.89 | 8.53 | 7.08 | | |
| | Dynaflect | 1000 | 1.37 | 1.26 | 1.11 | 0.95 | 0.79 | |
| 16 | FWD | 9000 | 11.33 | 10.08 | 8.33 | 7.18 | | |
| | Dynaflect | 1000 | 1.04 | 1.04 | 0.98 | 0.89 | 0.80 | |
| 17 | FWD | 9000 | 13.28 | 11.34 | 9.14 | 7.49 | | |
| | Dynaflect | 1000 | 0.81 | 0.73 | 0.62 | 0.50 | 0.40 | |
| 18 | FWD | 9000 | 12.56 | 11.01 | 8.88 | 7.74 | | |
| | Dynaflect | 1000 | 0.96 | 0.86 | 0.78 | 0.69 | 0.60 | |
| 19 | FWD | 9000 | 20.12 | 18.12 | 14.82 | 12.26 | | |
| | Dynaflect | 1000 | 0.92 | 0.77 | 0.65 | 0.54 | 0.44 | |
| 20 | FWD | 9000 | 11.78 | 10.56 | 8.68 | 7.51 | | |
| | Dynaflect | 1000 | 0.95 | 0.85 | 0.77 | 0.66 | 0.56 | |
| 21 | FWD | 9000 | 15.31 | 13.55 | 11.05 | 9.31 | | |
| | Dynaflect | 1000 | 1.04 | 0.96 | 0.83 | 0.69 | 0.53 | |

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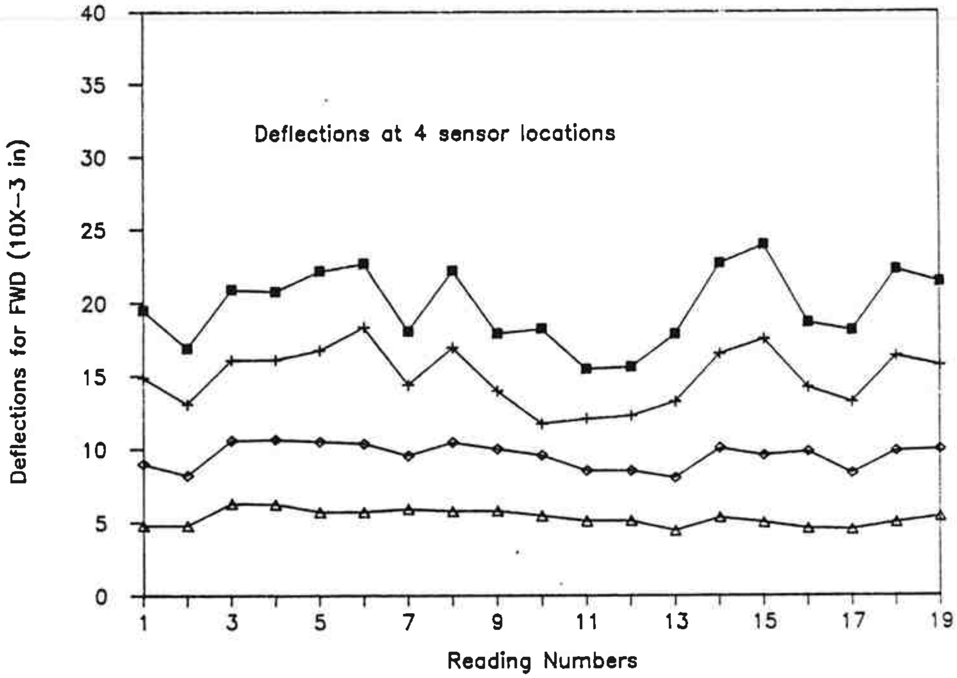
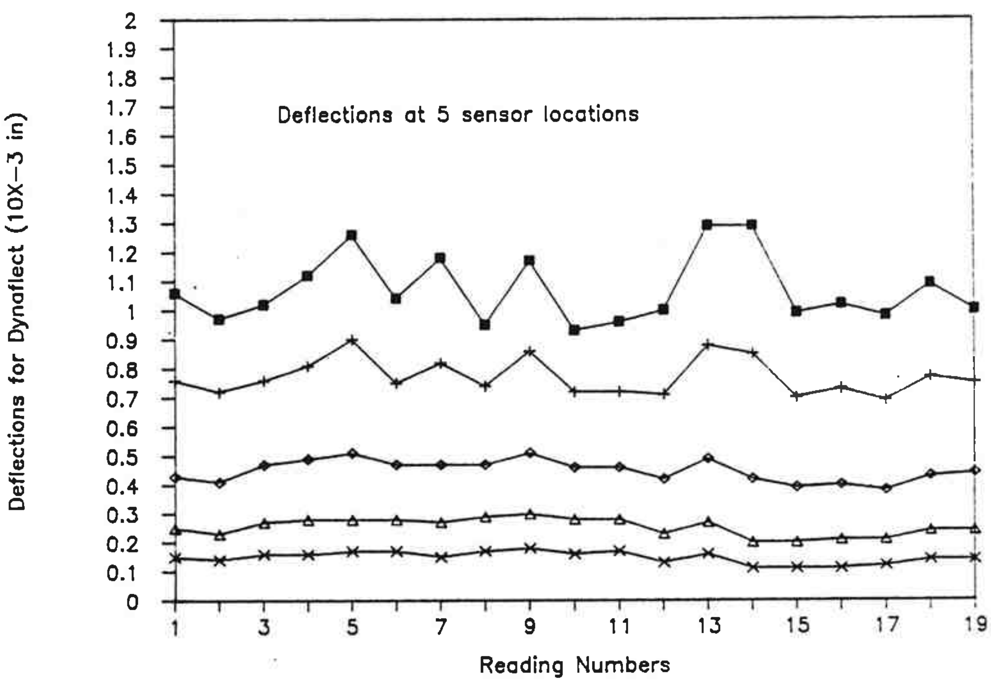
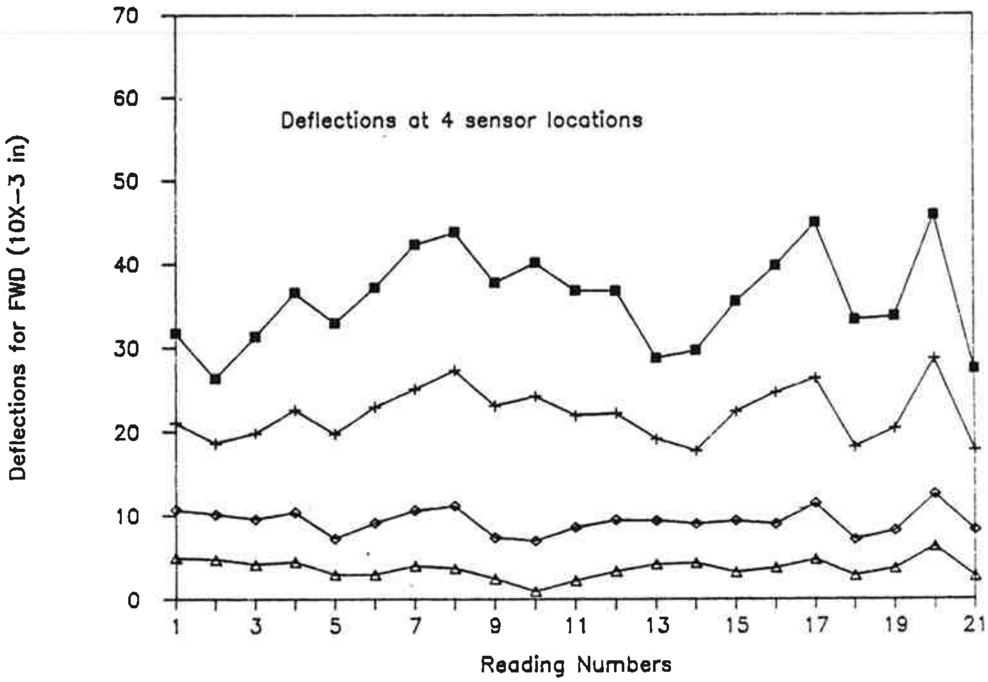



a) FWD

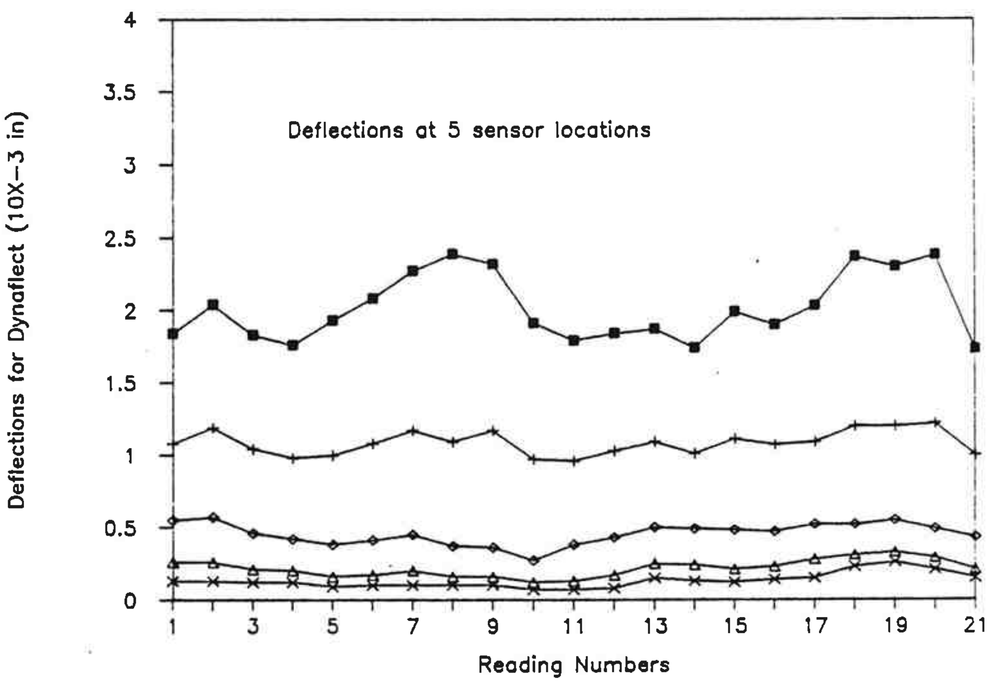



b) Dynaflect

Figure 6. Deflection Measurements for King's Valley Highway.

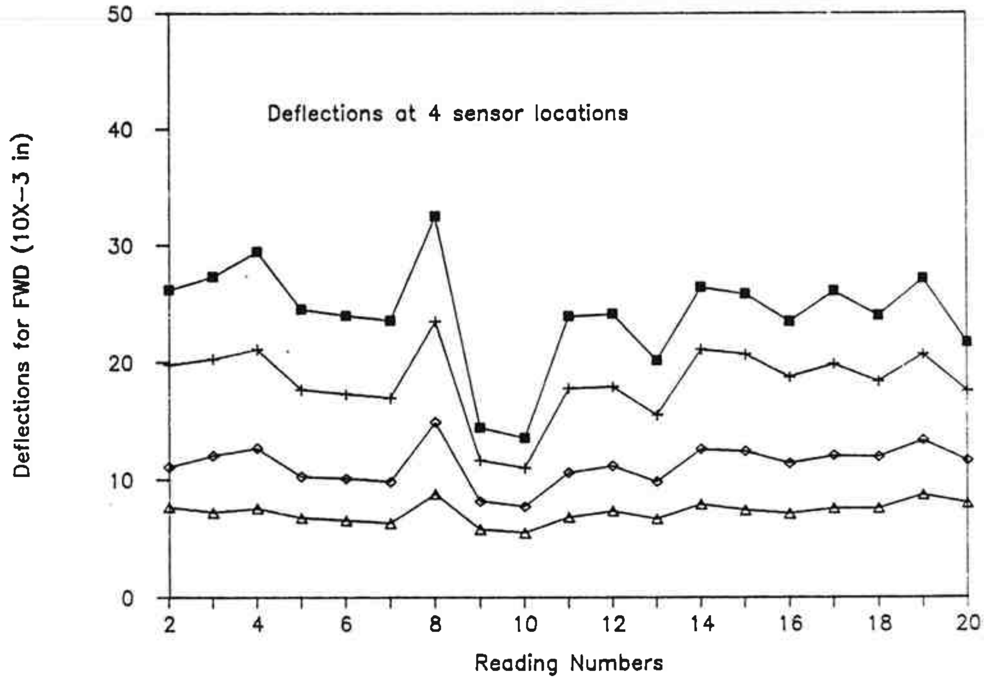



a) FWD

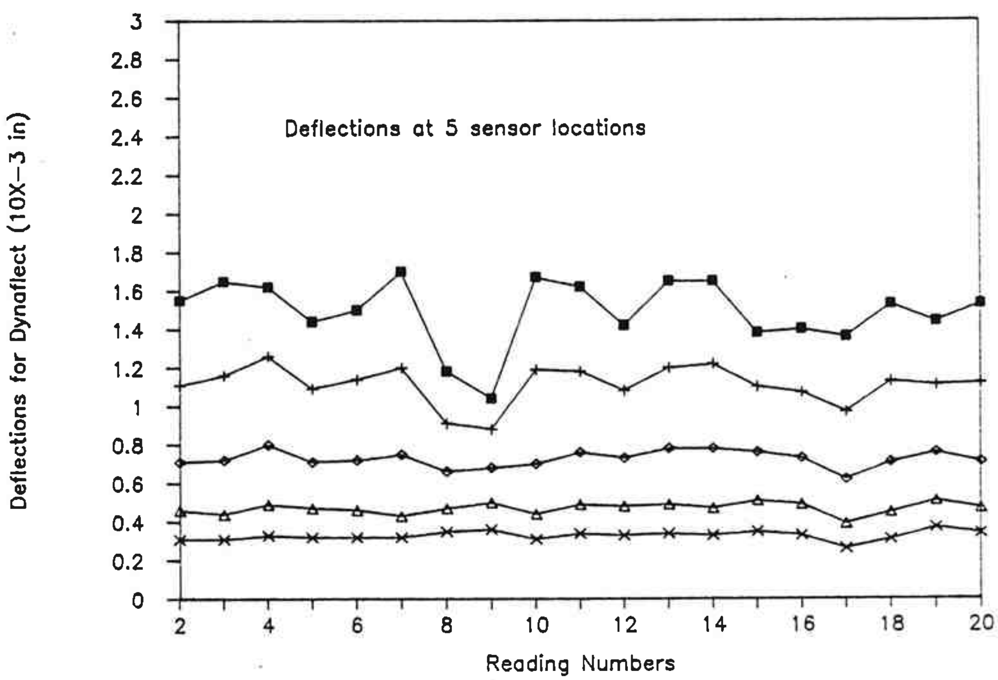



b) Dynaflect

Figure 7. Deflection Measurements for Willamina-Salem Highway.

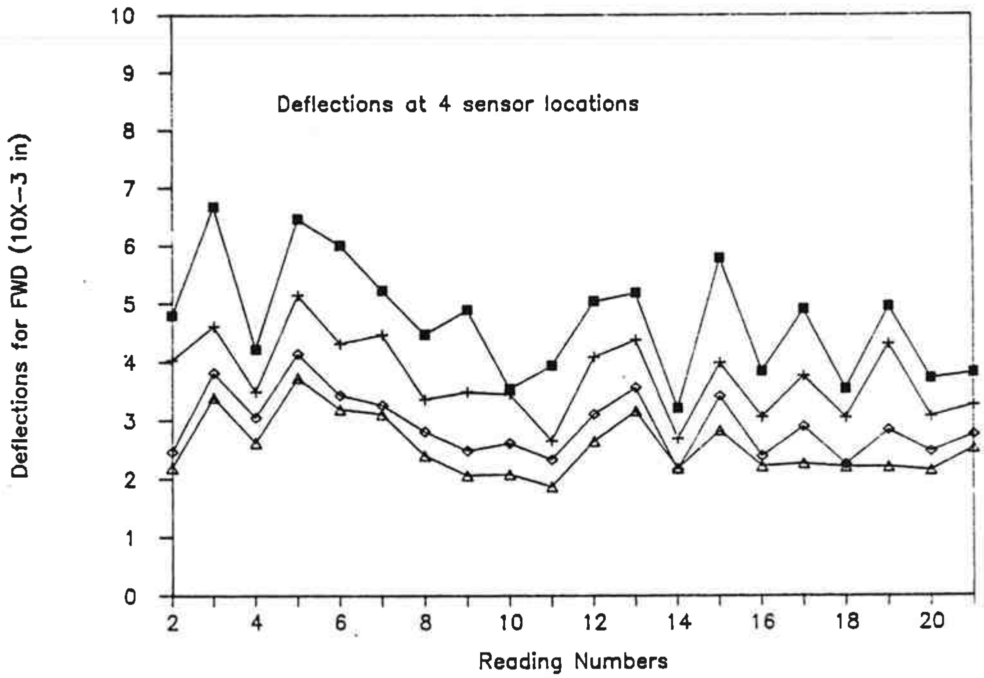



a) FWD

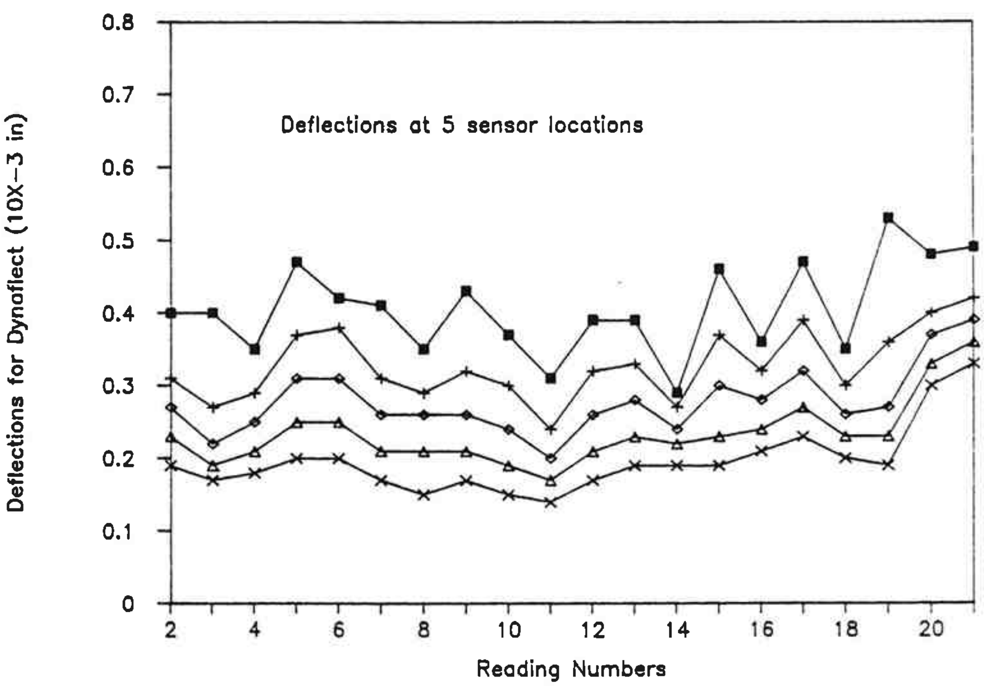



b) Dynaflect

Figure 8. Deflection Measurements for Lancaster Drive.

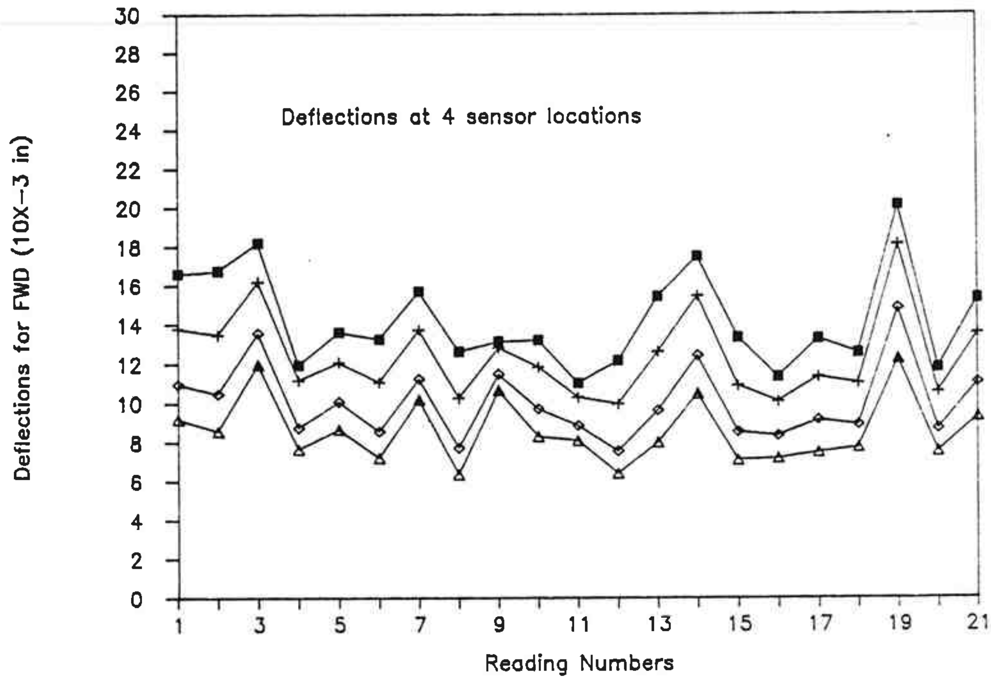



a) FWD

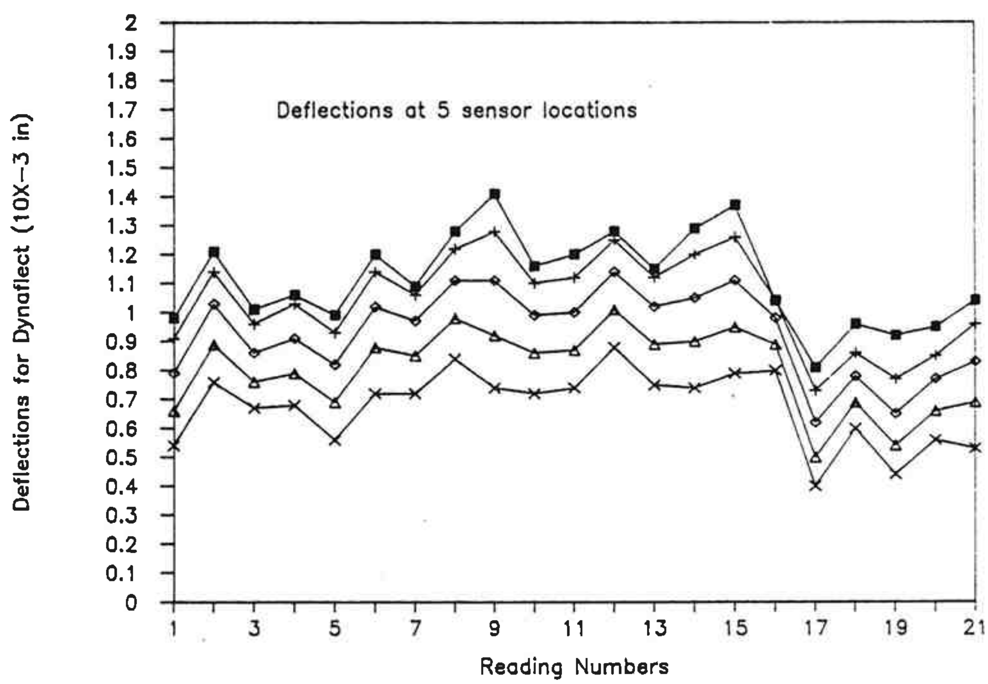



b) Dynaflect

Figure 9. Deflection Measurements for Salem Parkway.




a) FWD




b) Dynaflect

Figure 10. Deflection Measurements for Wilsonville-Hubbard Highway.

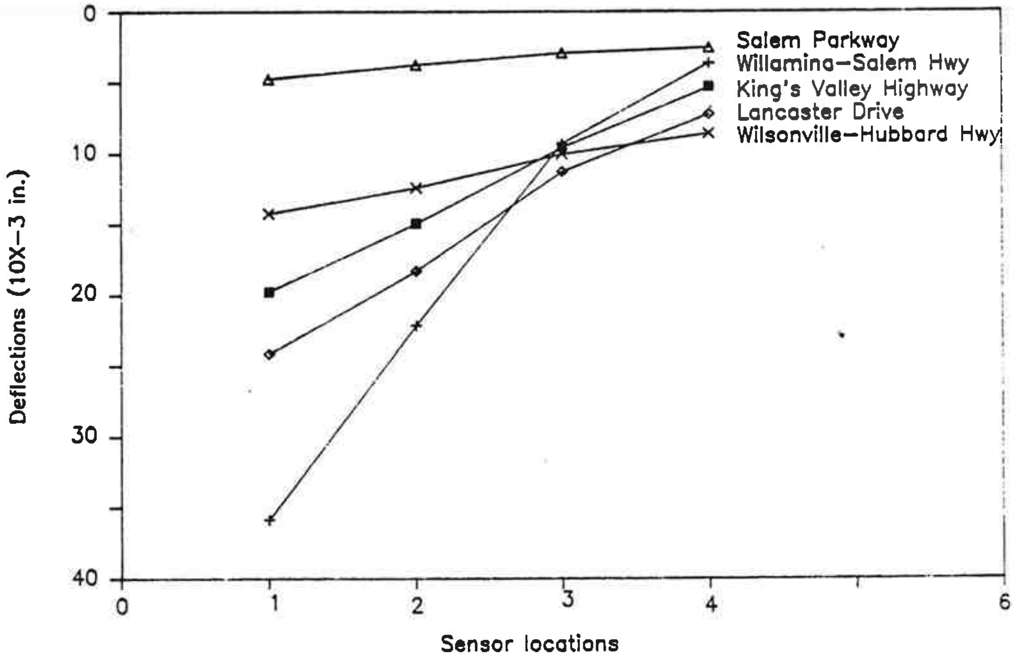



Figure 11. Average Deflection at Each Sensor Location (FWD).

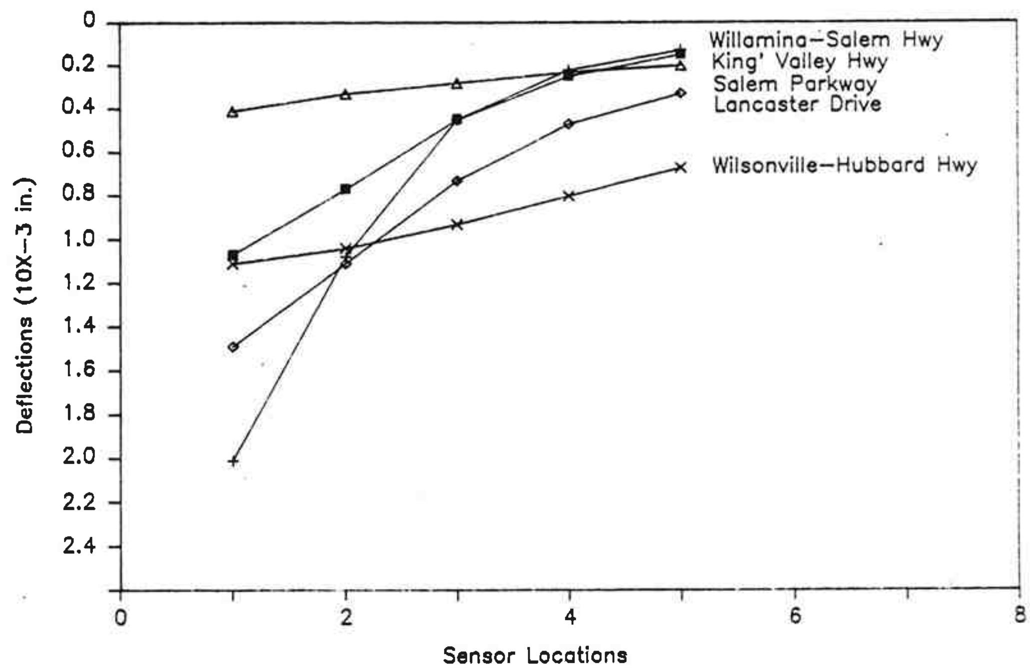



Figure 12. Average Deflection at Each Sensor Location (Dynaflect).

# 4.0 LABORATORY TESTS

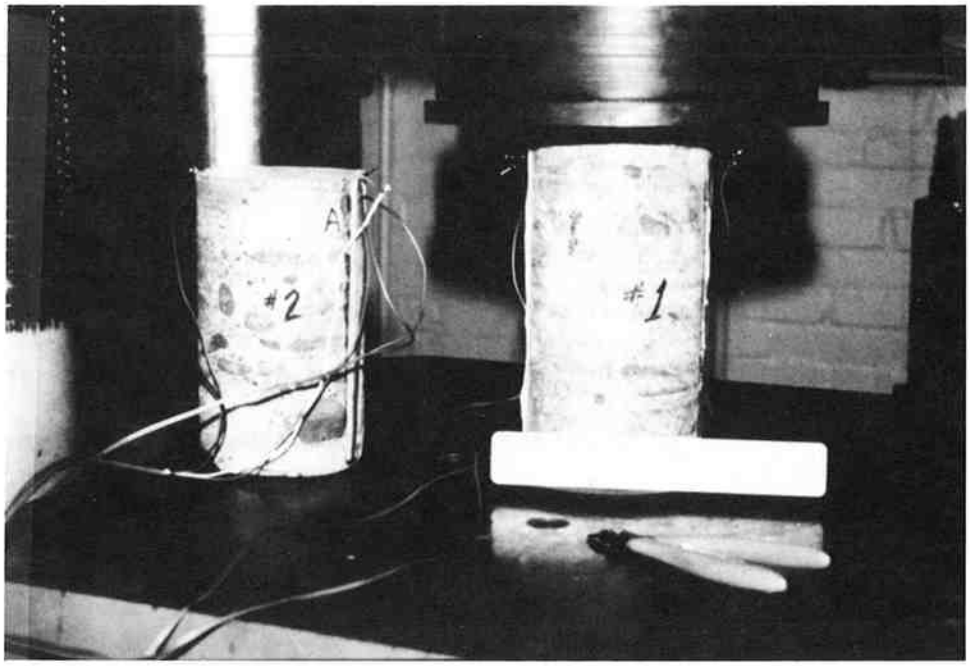
## 4.1 Test Procedures

Asphalt concrete and cement-treated base core samples (4-in. diameter) were obtained and tested in the laboratory for the resilient modulus. This test was done in accordance to ASTM D-4123. Sample preparation consisted of trimming the cores so that their heights were approximately 2.5 in. The unit weight of the materials were determined prior to testing. The testing procedure included placing the trimmed core into the diametral yoke and clamping it. The sample and yoke were then placed in the testing apparatus and aligned with both the bottom and top platens. The resilient modulus was determined at two test temperatures (10°C and 23°C) using a strain value ranging from 75 to 125 microstrain.

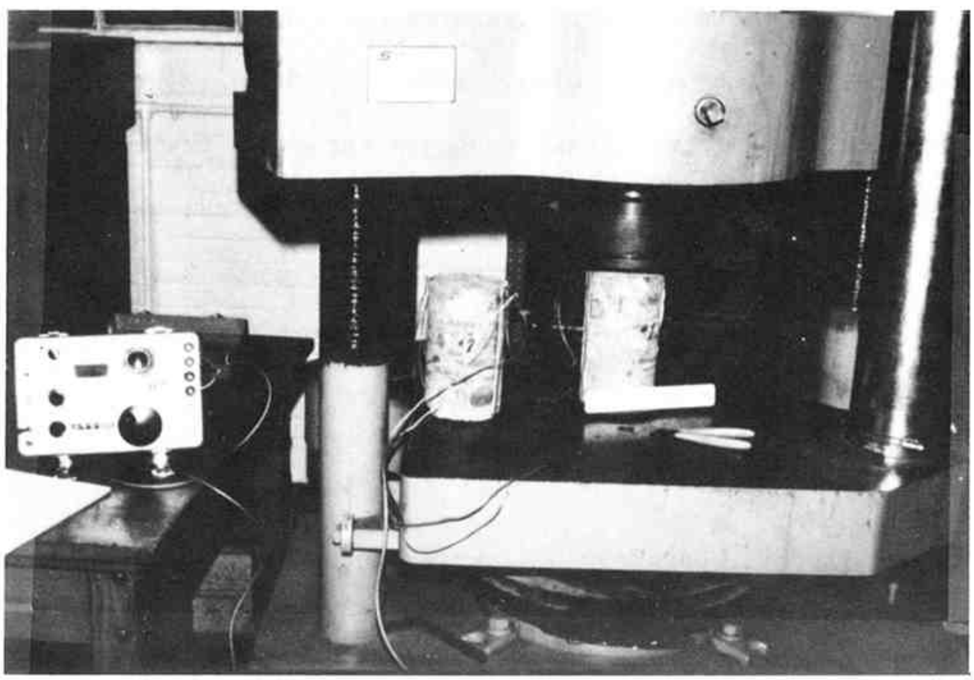
The resilient modulus of portland cement concrete cores were determined using 4-in. diameter by 8-in. high cylinders. The 4-in. cylinders were tested in compression using three strain gauges attached to the side of the specimen (Figure 13). A strain meter was used to detect the change in strain from changes in electrical resistance in the wire gauges. Strain values were recorded at several levels of load and tests were also conducted at 23°C and 10°C.

## 4.2 Results

The results of laboratory testing are presented in Table 7. The average modulus obtained was the result of testing the top (T), middle (M), and bottom (B) parts of the AC layer. This testing was possible only in situations where layer depth exceeded 2.5 in. by a substantial amount. Figure 14 shows the plot of modulus vs. temperature for each of the flexible pavements evaluated.




a) Test Cylinders



b) Strain Meter

Figure 13. Testing Apparatus for PCC Modulus.

Table 7. Resilient Modulus of Asphalt and Portland Cement Concrete Cores.

| Project | $M_R$ @ 23°C (psi) | | | $M_R$ @ 10°C (psi) | |
|---|---|---|---|---|---|
| King's Valley Highway | 608,000 | } T* | | 1,758,000 | |
| | 451,000 | } M* | Av = 568,000 | 1,409,000 | Av = 1,652,000 |
| AC | 375,000 | | | | |
| | 732,000 | } B* | | 1,791,000 | |
| | 673,000 | | | | |
| Willamina-Salem Highway | 320,000 | } ** | | 1,162,000 | |
| | 307,000 | | | | |
| | 396,000 | } ** | Av = 346,000 | 1,369,000 | Av = 1,272,000 |
| AC | 334,000 | } ** | | 1,286,000 | |
| | 306,000 | | | | |
| Lancaster Drive | 264,000 | } T* | | 1,045,000 | |
| | 275,000 | | | | |
| AC | 336,000 | } M* | Av = 403,000 | 801,000 | Av = 1,242,000 |
| | 297,000 | | | | |
| | 843,000 | } B* | | 1,881,000 | |
| Salem Parkway | 217,000 | } ** | | 760,000 | |
| AC | 200,000 | | Av = 231,750 | | Av = 1,149,000 |
| | 257,000 | } ** | | 1,538,000 | |
| | 253,000 | | | | |
| Wilsonville-Hubbard Highway | 5,891,300 | | | | |
| | 4,064,700 | | Av = 4,977,000 | N/A | |
| PCC | | | | | |
| Salem Parkway | 1,520,000 | | | | |
| | 2,000,000 | | Av = 1,808,000 | N/A | |
| CTB | 1,894,000 | | | | |

*T = Top; B = Bottom; M = Middle.
**Samples tested at two strain levels.1

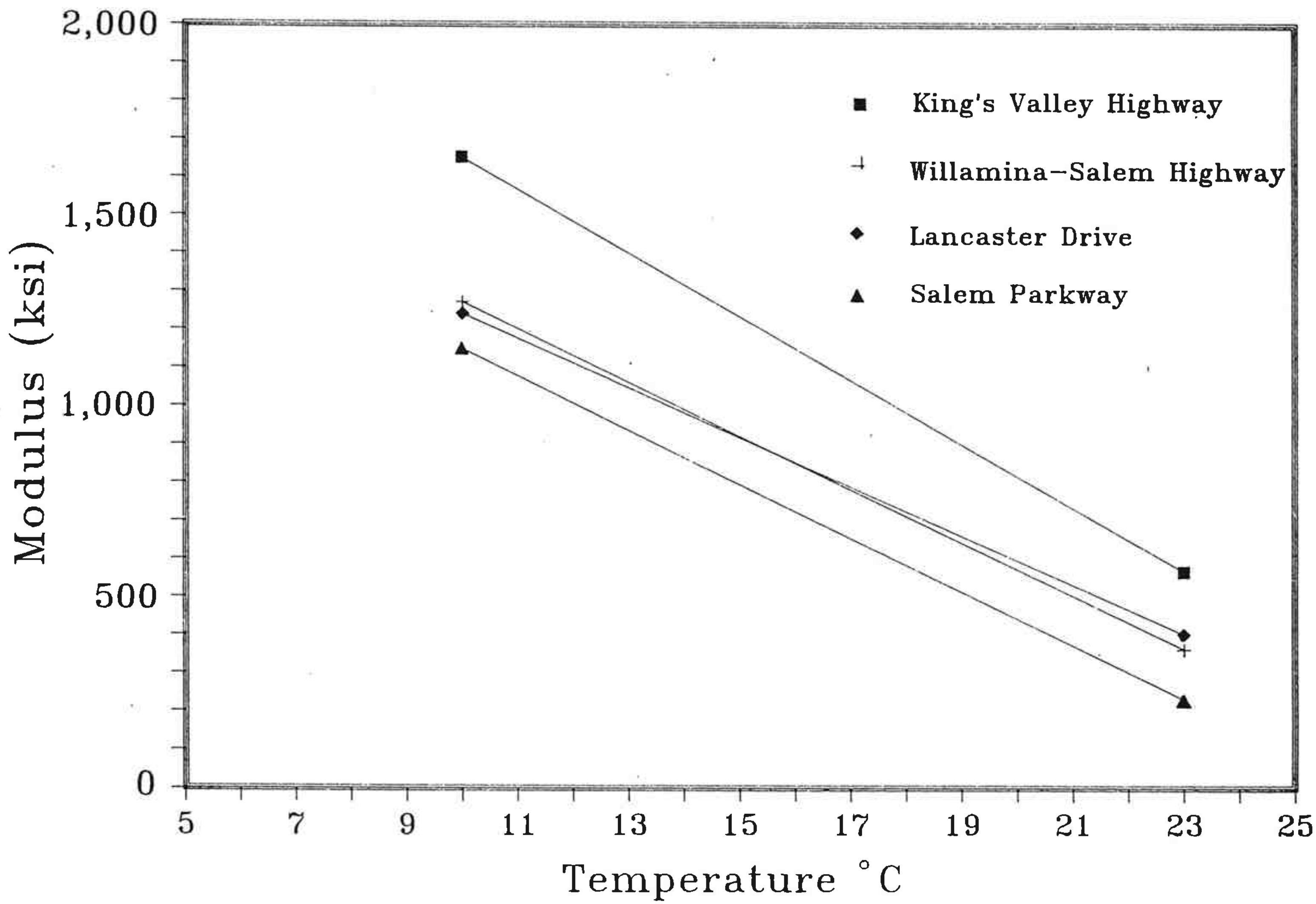



Figure 14. Plot of Modulus vs. Temperature.

Note there were considerable differences in the modulus values between projects.

The results for the cement-treated base indicated a modulus of (~2,000,000 psi) while those for the portland cement concrete were slightly less than 5,000,000 psi.

## 5.0 DETERMINING THE EXISTING PAVEMENT STRENGTH

The structural capacity of the existing pavements was estimated by using both NDT methods 1 and 2 of the AASHTO Guides (5). Three backcalculation programs (BISDEF, ELSDEF, and MODCOMP2) were utilized for NDT method 1. For NDT method 2, calculations were carried out by following the procedures described in AASHTO Guide (5). The following paragraphs describe briefly the methodologies that have been used in this study and present results obtained from both NDT method 1 and NDT method 2.

### 5.1 NDT Method 1

NDT method 1 is based upon analysis of the deflection basin data obtained from the NDT testing device. Backcalculation is necessary in determining layer moduli of pavements. Three programs were used and each of them is briefly described below.

#### 5.1.1 BISDEF

This computer program was developed by the U.S. Army Corps of Engineers, Waterways Experiment Station (8,9). It uses the deflection basin from nondestructive testing (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.

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:

1) thickness of each layer,
2) range of allowable modulus,
3) initial estimate of modulus, and

4) Poisson's ratio.

For the deflection basin, the required inputs are:

1) load and load radius of a NDT testing device,

2) deflections at a number of sensor locations, and

3) 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 testing. The number of layers with unknown modulus values cannot exceed the number of measured deflections. The best results may be obtained when less variable layers for moduli need to be calculated.

The program is solved using an iterative process which 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 sub-routine 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.

### 5.1.2 ELSDEF

The ELSDEF program is a modification of the program BISDEF (10). The modification was performed by Brent Rauhut Engineers and uses the computer program (ELSYM5) developed at the University of California at Berkeley (13). It determines the various component stresses, strains, and displacements along with principal values in a three-dimensional ideal elastic-layered system. The layered system can be loaded with one or more identical uniform circular loads normal to the surface of the system.

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.

### 5.1.3 MODCOMP2

MODCOMP2 was developed by Irwin (11) of Cornell University. The program specifications include the following:

1) Up to eight layers can be included in the pavement system.
2) The layer combinations may be linear elastic or nonlinear stress-dependent.
3) It is capable of accepting data from several typical NDT devices (e.g., FWD, Road Rater, and Dynaflect).
4) It is capable of accepting up to six load levels.

MODCOMP2 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 which 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 deflection is within the specified tolerance or until the maximum number of iterations has been reached.

Since untreated base course and subgrade materials behave as nonlinear materials, the resilient modulus of such materials can be expressed by the following equation,

$$M_r = K_1 \theta^{K_2}$$

where $\Theta$ is bulk stress and $K_1$ and $K_2$ are constants. The program also has the added capability to derive the $K_1$ and $K_2$ parameters when they are unknown for a given layer. In such cases, the user must provide deflection basin data for at least three different load levels. The program can accept data for up to six different load levels.

### 5.1.4 Backcalculation Results

Tables 8 to 12 show the results of backcalculation using both the FWD and Dynaflect data on all five project sites. For each project site, five test locations were selected for analysis. The selection of deflection basin was based on whether the measured deflection at load center is between the mean value and the mean value plus 1.5 standard deviations for that particular road section. The backcalculation was carried out using the above three programs with three different procedures in an attempt to obtain consistent results. Procedure 1 uses a fixed surfacing modulus for each project site to determine moduli of the base and subgrade. The surfacing modulus was determined from the laboratory test. Procedure 2 uses both a fixed surfacing 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:

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

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 (inches) from the plate load center, and

Table 8. 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. 14).
**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. 14).
**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 9. 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. 14).
**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. 14).
**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 10. 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. 14).
**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. 14).
**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 11. 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. 14).

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. 14).
**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 12. Backcalculated Moduli (psi) for Wilsonville-Hubbard Highway.

a) Procedure 1 - Fixed Surface Modulus

| Location Identification | Layer* | FWD BISDEF | FWD ELSDEF | FWD MODCOMP2 | Dynaflect BISDEF | Dynaflect ELSDEF | Dynaflect 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 BISDEF | FWD ELSDEF | FWD MODCOMP2 | Dynaflect BISDEF | Dynaflect ELSDEF | Dynaflect 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 BISDEF | FWD ELSDEF | FWD MODCOMP2 | Dynaflect BISDEF | Dynaflect ELSDEF | Dynaflect 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: $E_{sg} = (PS_f)/(rd_r)$.

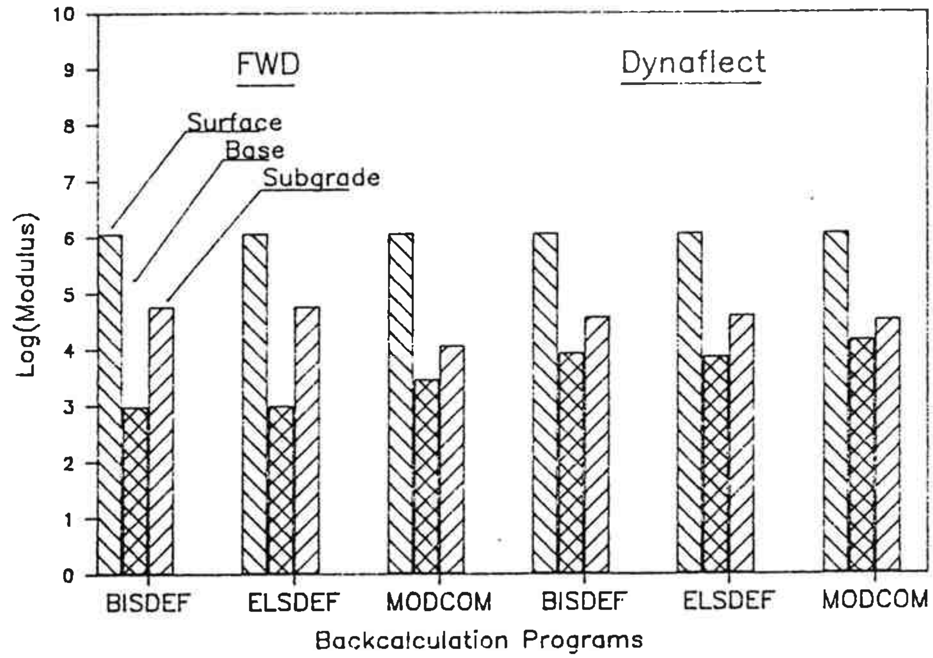
$S_f$ = subgrade modulus prediction factor, which is a function of radius of NDT load plate, Poisson's ratio, and pavement's effective thickness.

Procedure 3 uses the preestimated subgrade modulus alone to solve for the surface and base layer moduli.

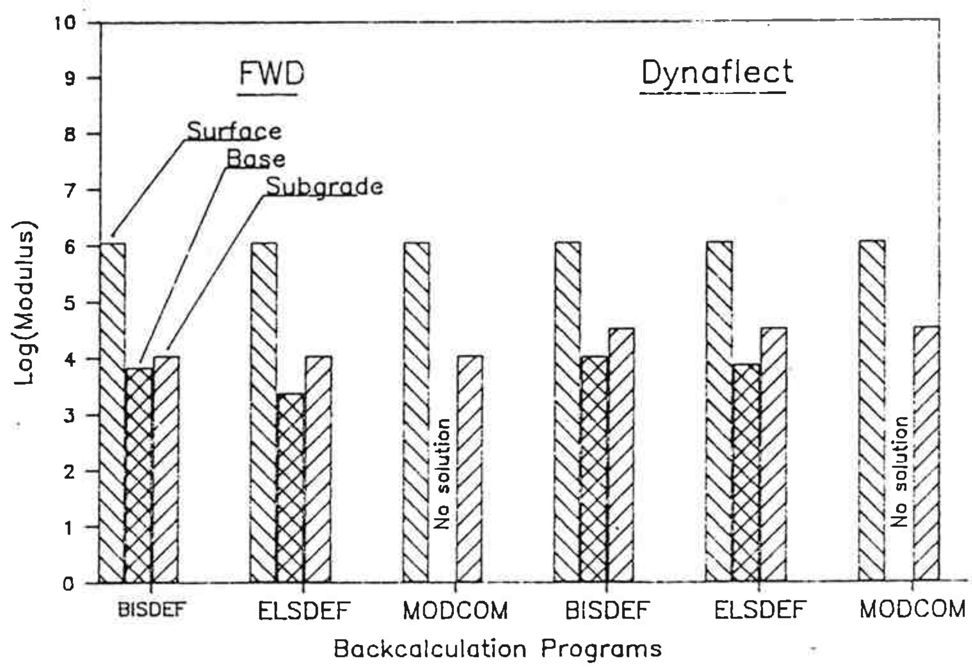
Procedure 1. With this procedure, the surfacing modulus for each project was determined from laboratory tests and used as a fixed value in the backcalculation. For Salem Parkway, base and subbase were treated as one layer, thus eliminating one variable for determining modulus. The results from three programs using FWD data show that King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive sites have a very weak base layer, while 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. Subgrade moduli are constantly higher in all 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 both those of ELSDEF and MODCOMP2. With Dynaflect data, ELSDEF presents highest modulus values among three programs. In all cases, MODCOM2 provides lowest values, varies from 40 to a few hundred percent lower than those of BISDEF and ELSDEF. Subgrade modulus values calculated from BISDEF and MODCOMP2 are relatively close for each NDT testing device, while ELSDEF gives consistent lower modulus value using both FWD and Dynaflect data. Wilsonville-Hubbard Highway is a PCC pavement. Its surfacing layer modulus is about 5,000,000 psi as tested in laboratory while 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 uses 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 be seen easily. For all four flexible pavements,the program BISDEF presents constant higher modulus than ELSDEF and MODCOMP2 as can be seen in Figures 15b to 18b. The results of using this method show many similarities with 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 solution in some test locations.

Procedure 3. The third procedure uses an estimated subgrade modulus as a fixed input to solve for surface and base moduli. The results are presented in Tables 8c to 12c and shown in Figures 15c to 19c. Although the backcalculated moduli vary for each program, with MODCOMP2 giving consistently higher surfacing layer modulus using Dynaflect data, the results from each individual program are fairly close for the projects at King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive. As one would expect the Willamina-Salem Highway has the lowest modulus values since it had the highest measured deflection at NDT device load center (Figures 11 and 12). The King's Valley

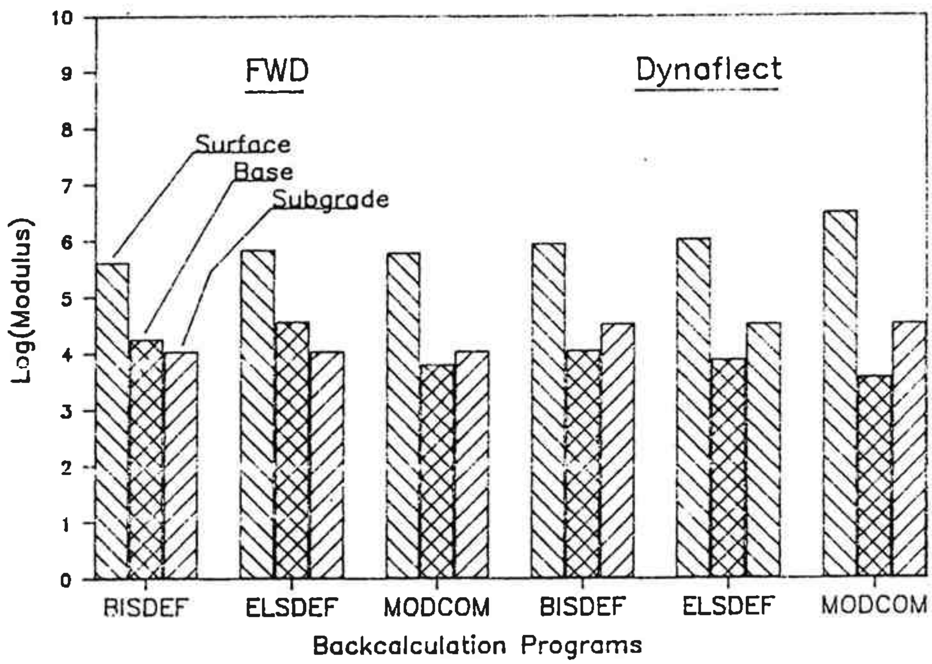



a) Procedure 1



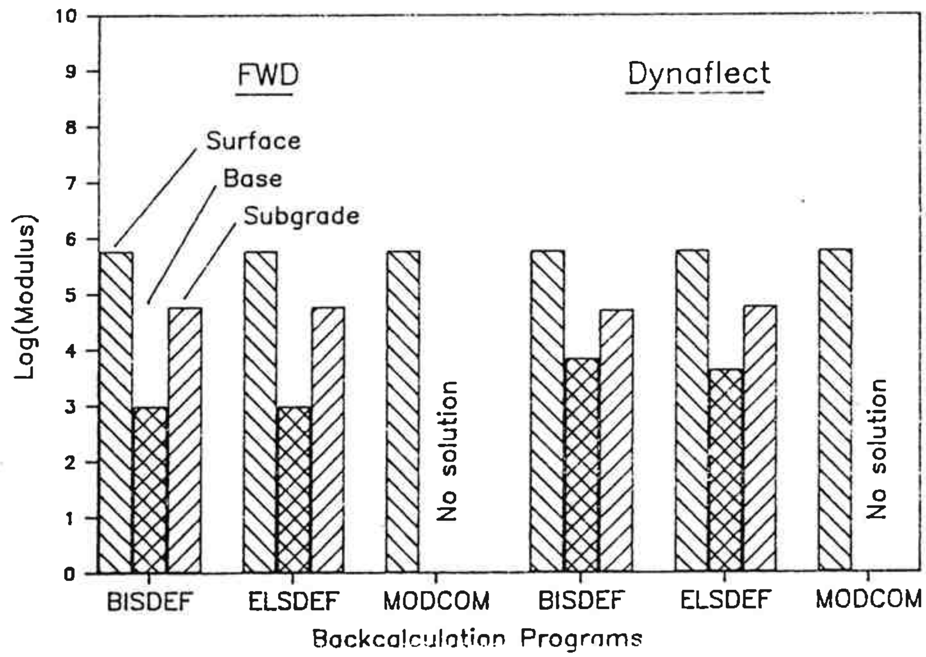

b) Procedure 2

Figure 15. Backcalculated Moduli for King's Valley Highway.

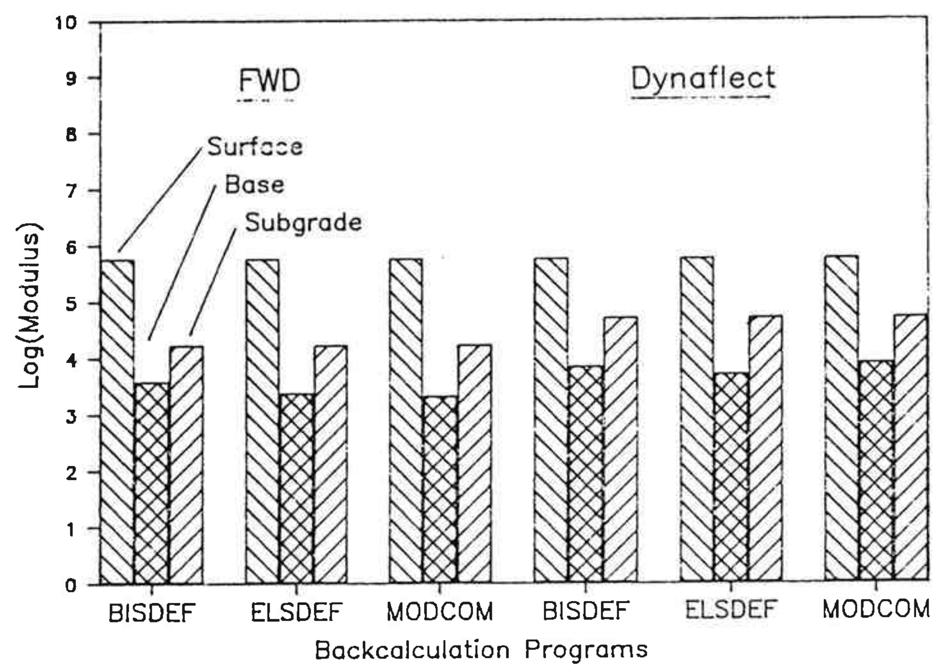



c) **Procedure** 3

Figure 15. Backcalculated Moduli for King's Valley Highway (Continued).

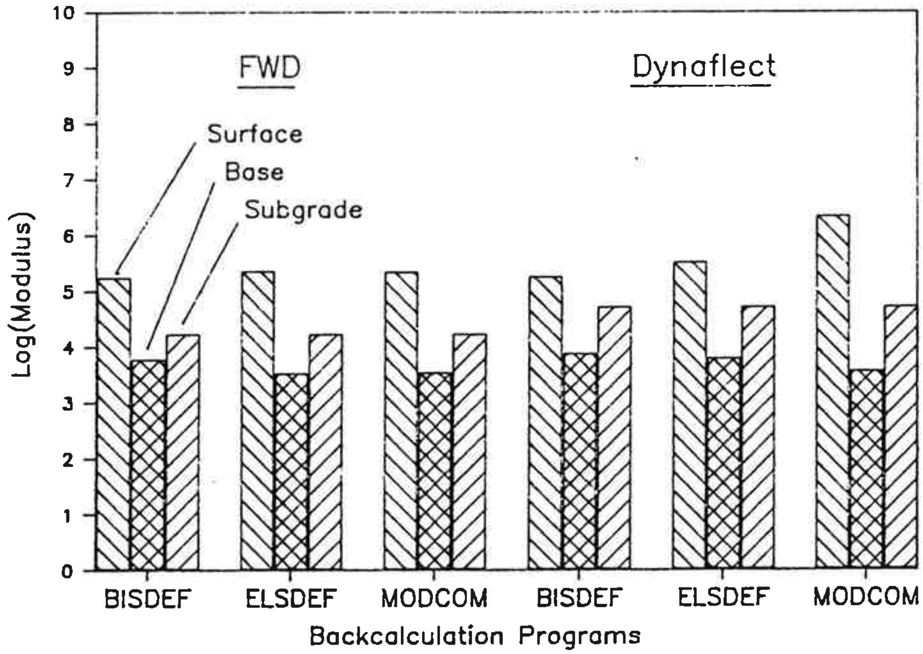



a) Procedure 1



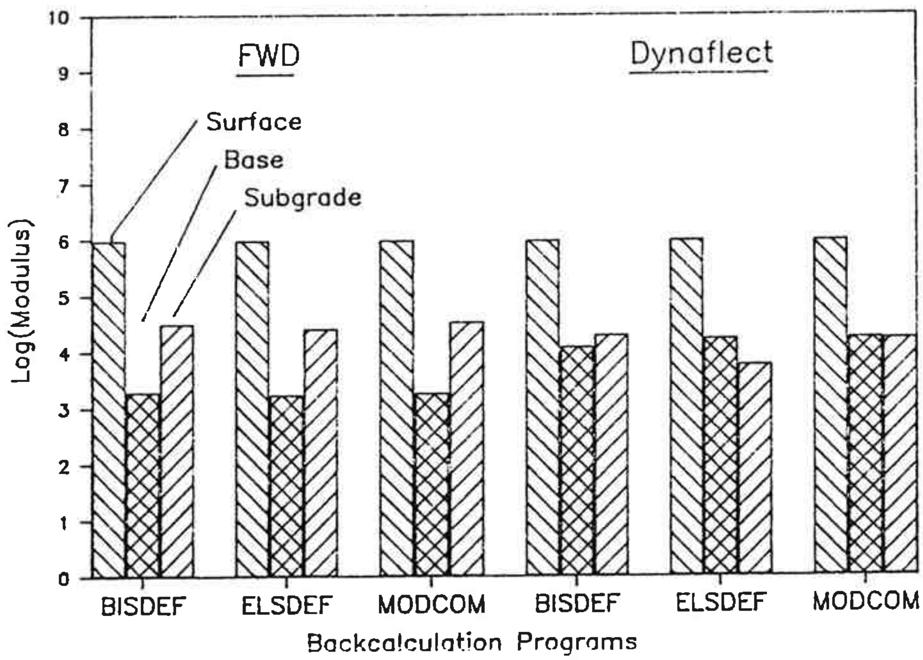

b) Procedure 2

Figure 16. Backcalculated Moduli for Willamina-Salem Highway.

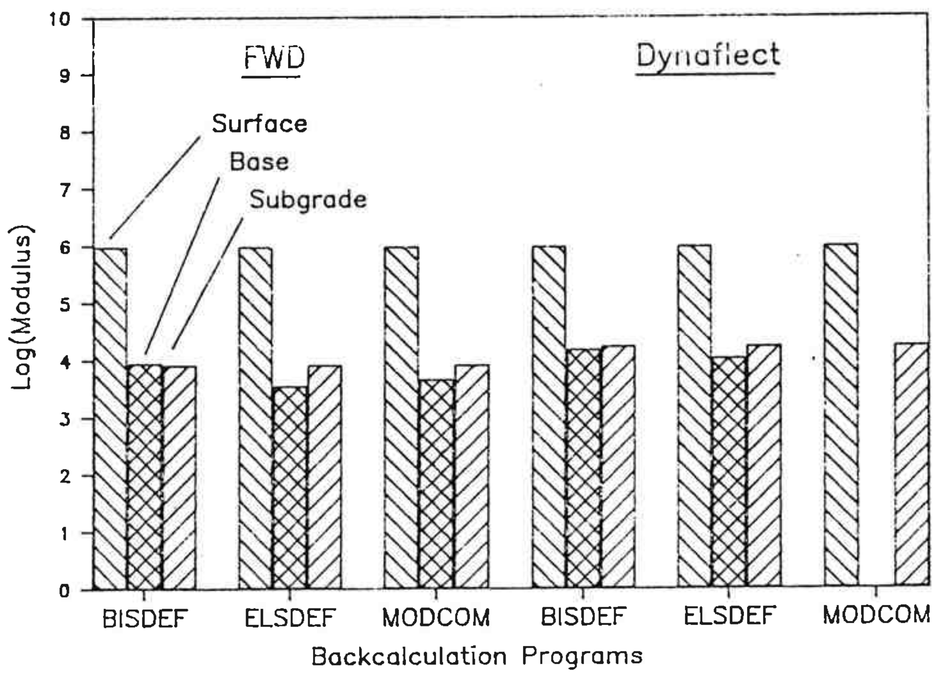



c) Procedure 3

Figure 16. Backcalculated Moduli for Willamina-Salem Highway (Continued).

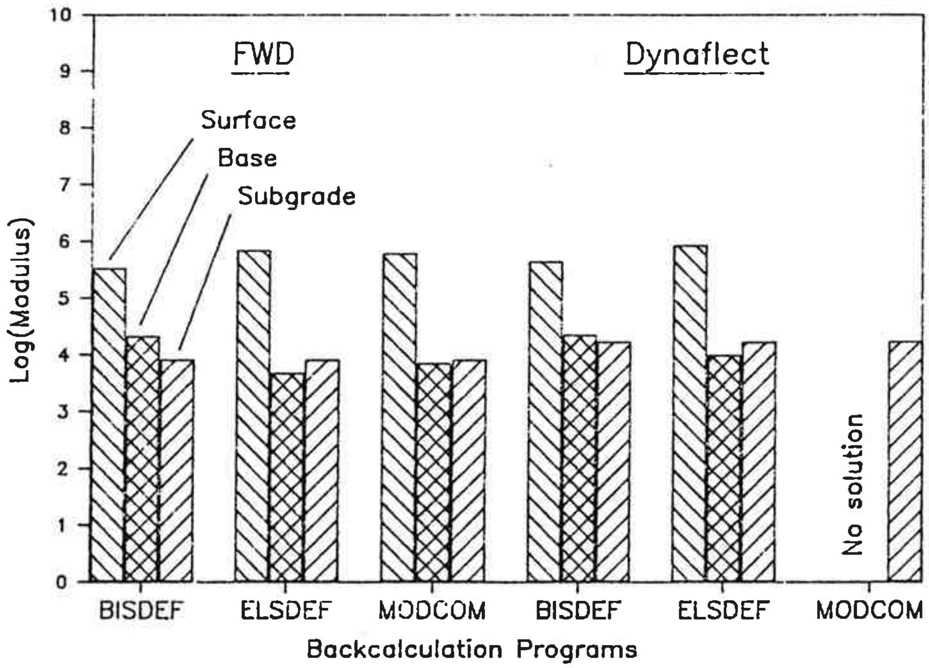



a) Procedure 1



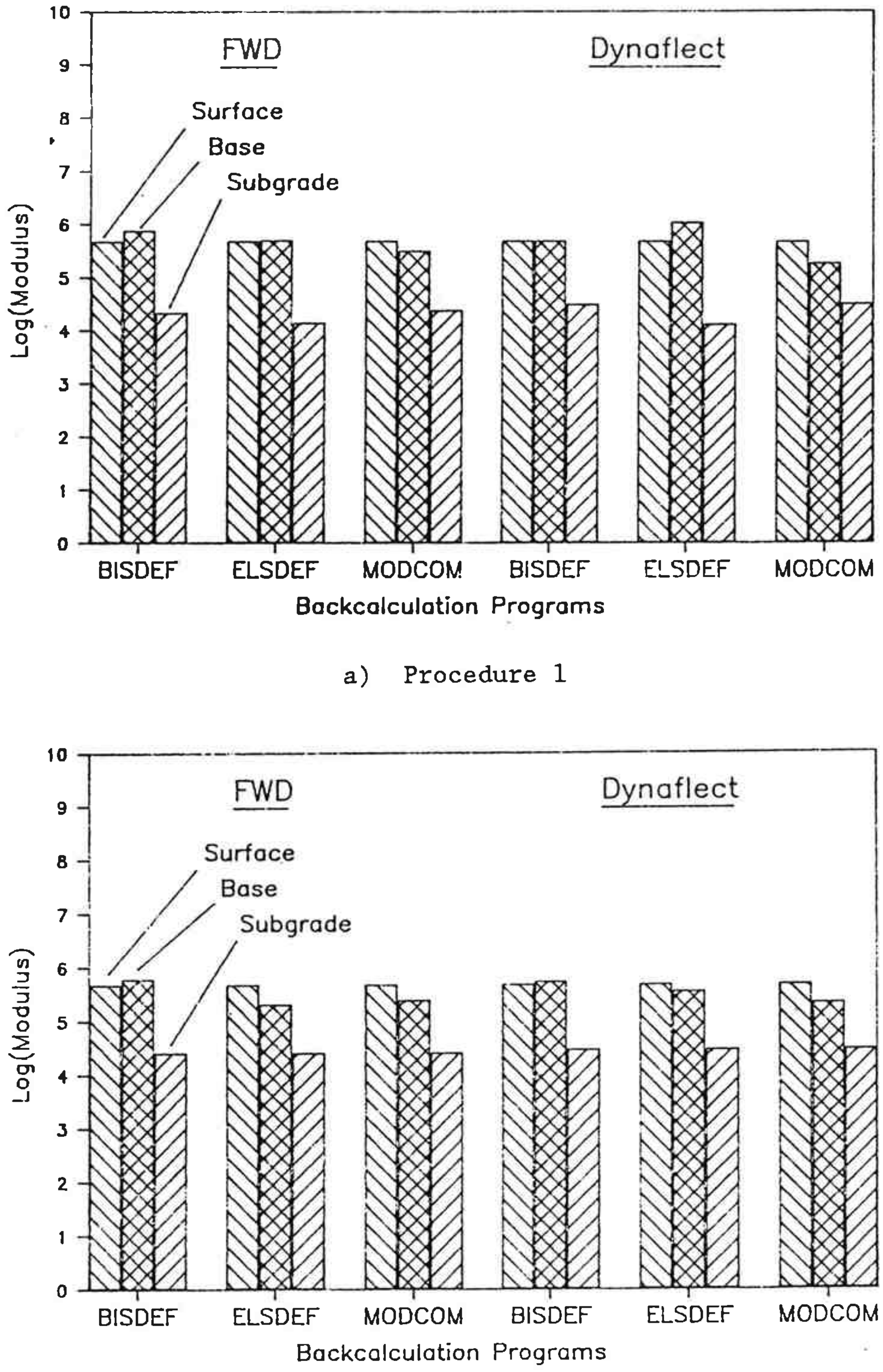

b) Procedure 2

Figure 17. Backcalculated Moduli for Lancaster Drive.




c) Procedure 3

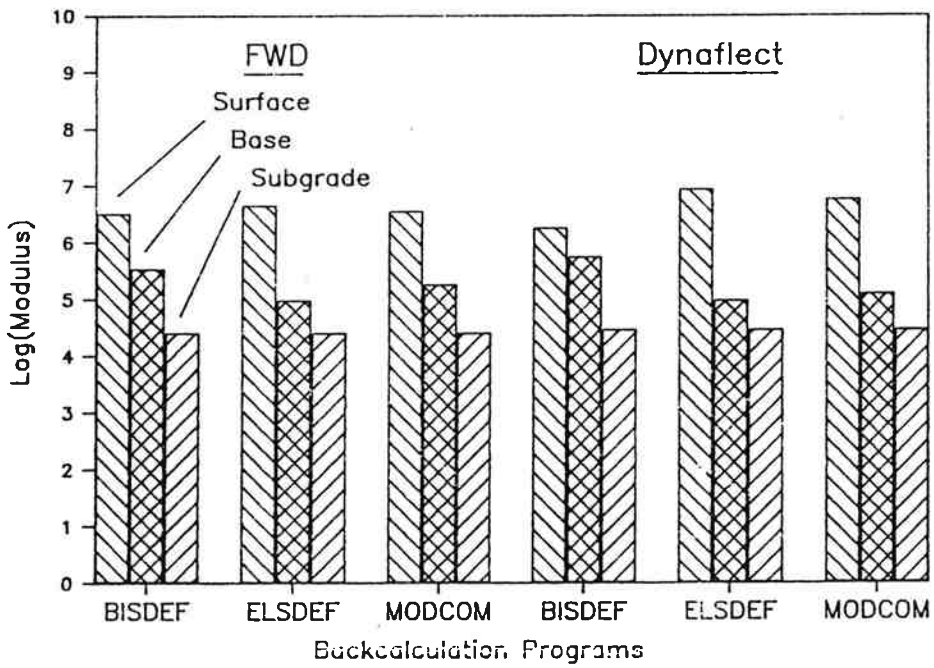
Figure 17. Backcalculated Moduli for Lancaster Drive (Continued).




a) Procedure 1

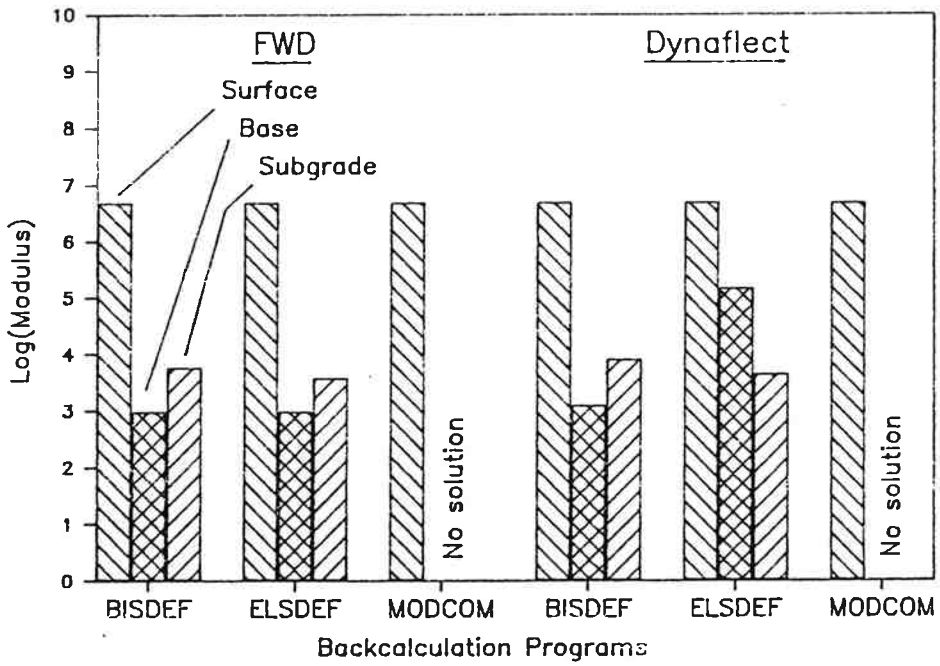
b) Procedure 2

Figure 18. Backcalculated Moduli for Salem Parkway.

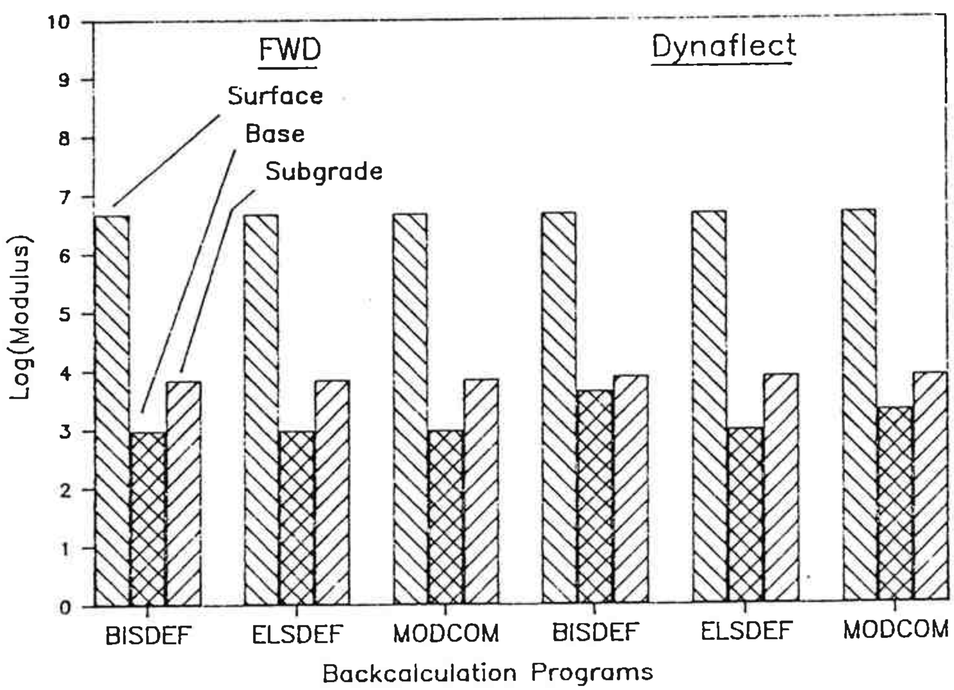



c) **Procedure 3**

Figure 18. Backcalculated Moduli for Salem Parkway (Continued).

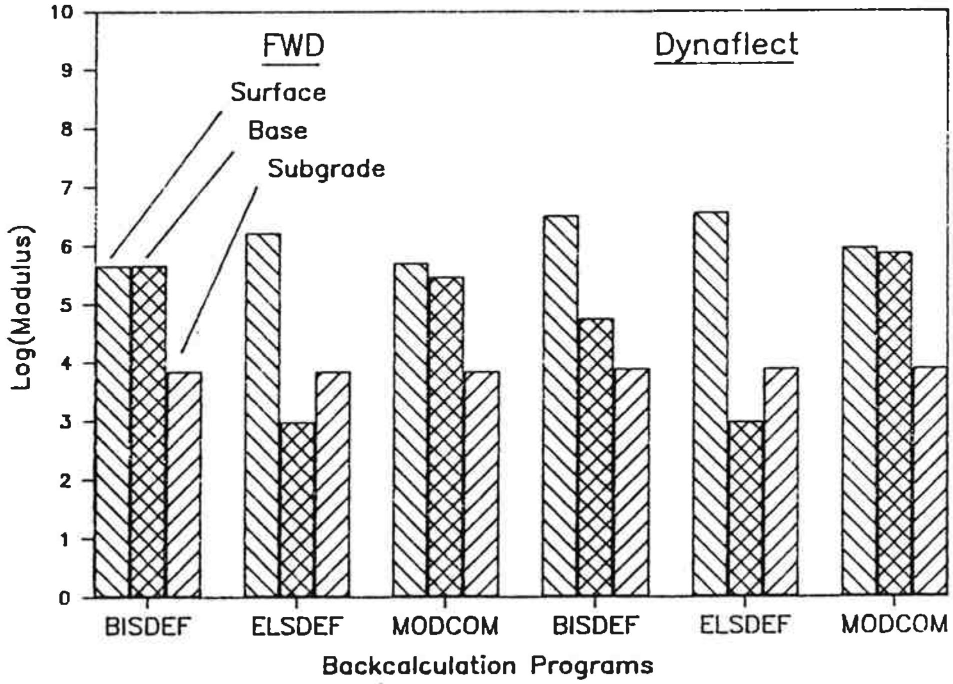



a) Procedure 1




b) Procedure 2

Figure 19. Backcalculated Moduli for Wilsonville-Hubbard Highway.




c) Procedure 3

Figure 19. Backcalculated Moduli for Wilsonville-Hubbard Highway (Continued).

Highway has the highest modulus because of its smaller deflection readings. While Lancaster Drive is in between. The backcalculated results reflect this phenomena very well. For Willamina-Salem Highway, using BISDEF results, the average modulus for the surface is about 180 ksi. For King's Valley Highway, the average surface modulus is close to 440 ksi and approximately 350 ksi for Lancaster Drive. The backcalculated moduli for base layer 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 Figure 18c. This fact is also reflected in 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.

## 5.2 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 $SN_{xeff}$ from Burmister's two-layer deflection theory. The relationship between deflection and structural number is given by the following:

$$d_o = \left[\frac{2P(.0043*h_t)^3}{3.1416\ a_c SN^3}\right]\left[1 + F_b\left[\frac{SN^3(1-\mu_{sg})^2}{E_{sg}(.0043*h_t)^3} - 1\right]\right]$$

where: $d_o$ = deflection value

$P$ = NDT device load (in lbs)

$h_t$ = total layer thickness (above subgrade)

$\mu_{sg}$ = subgrade Poisson's ratio

$E_{sg}$ = subgrade modulus

SN = $SN_{xeff}$

$F_b$ = Boussinesq one layer deflection factor and is given by

$$F_b = \left[\left[1 + \left[\frac{h_e}{a_c}\right]^2\right]^{.5} - \frac{h_e}{a_c}\right] \left[1 + \frac{(h_e/a_c)}{2(1-\mu_{sg})\left[1 + \left(\frac{h_e}{a_c}\right)^2\right]^{.5}}\right]$$

where: $h_e$ = equivalent transformed thickness and is expressed as

$$h_e = 0.9\ h_t \left[\frac{E_e(1-\mu_{sg}^2)}{E_{sg}(1-\mu_e^2)}\right]^{1/3}$$

$\mu_e$ can be selected to have any value, so if it is assumed that $\mu_e$ is equal to $\mu_{sg}$, then

$$h_e = 0.9\ h_t \left[\frac{E_e}{E_{sg}}\right]^{1/3}$$

$E_e$ = equivalent pavement modulus and is given by:

$$E_e = \left[\frac{SN}{.0043*h_t}\right]^3 (1 - \mu_{sg}^2)$$

The $SN_{xeff}$ value for a particular pavement structure 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 (14).

### 5.3 Existing Pavement Structural Capacity

The structural capacity of the existing pavements was determined using both NDT method 1 and NDT method 2. For NDT method 1, the $SN_{xeff}$ values for each test location were computed for both FWD and Dynaflect using the backcalculated results from the three programs. For King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive, backcalculated moduli from procedure 3 were used to determine the $SN_{xeff}$, while for Salem-Parkway and Wilsonville-Hubbard highway, results from procedure 1 were used. The calculated $SN_{xeff}$ are shown in Tables 13 to 17. The maximum surfacing layer coefficient was set at 0.44 for all four flexible pavement projects. The layer coefficients for the base were determined based upon the modulus values calculated from the backcalculation programs (see Appendix B for calculations).

For NDT method 2, the $SN_{xeff}$ values were determined using the procedures described in Section 5.2, while the subgrade modulus was estimated using a method presented in Section 5.1.4. The results for both FWD and Dynaflect are presented in Tables 18 to 22.

The results generally indicate the following:

1) The $SN_{xeff}$ calculated from BISDEF results using FWD data are generally higher than those of ELSDEF and MODCOMP2 except for Willamina-Salem Highway.

2) For NDT method 2, the calculated $SN_{xeff}$ using Dynaflect data are consistently higher than that of using FWD data.

Table 13. Calculated $SN_{xeff}$ for King's Valley Highway (NDT Method 1).

| Location Identification | FWD BISDEF | FWD ELSDEF | FWD MODCOMP2 | Dynaflect BISDEF | Dynaflect ELSDEF | Dynaflect MODCOMP2 |
|---|---|---|---|---|---|---|
| 3 | 4.32 | 2.78 | 2.78 | 3.20 | 2.78 | 2.78 |
| 4 | 4.18 | 2.78 | 2.78 | 2.92 | 2.78 | 2.78 |
| 5 | 3.70 | 2.78 | 2.78 | 2.92 | 2.78 | 2.78 |
| 8 | 3.70 | 2.78 | 2.78 | 3.48 | 2.78 | 2.78 |
| 18 | 3.30 | 2.78 | 2.78 | 3.34 | 2.78 | 2.78 |
| Average | 3.84 | 2.78 | 2.78 | 3.17 | 2.78 | 2.78 |

Table 14. Calculated $SN_{xeff}$ for Willamina-Salem Highway (NDT Method 1).

| Location Identification | FWD BISDEF | FWD ELSDEF | FWD MODCOMP2 | Dynaflect BISDEF | Dynaflect ELSDEF | Dynaflect MODCOMP2 |
|---|---|---|---|---|---|---|
| 4 | 1.77 | 2.04 | 1.93 | 1.95 | 2.35 | 2.51 |
| 7 | 1.61 | 1.82 | 1.82 | 1.51 | 2.09 | 2.51 |
| 8 | 1.61 | 1.82 | 1.82 | 1.35 | 1.98 | 2.51 |
| 12 | 1.77 | 1.98 | 1.88 | 2.04 | 2.25 | 2.51 |
| 16 | 1.61 | 1.82 | 1.88 | 1.90 | 2.35 | 2.51 |
| Average | 1.67 | 1.90 | 1.87 | 1.75 | 2.20 | 2.51 |

Table 15. Calculated $SN_{xeff}$ for Lancaster Drive (NDT Method 1).

| Location Identification | FWD | | | Dynaflect | | |
|---|---|---|---|---|---|---|
| | BISDEF | ELSDEF | MODCOMP2 | BISDEF | ELSDEF | MODCOMP2 |
| 2 | 4.27 | 2.60 | 2.60 | 4.24 | 2.78 | N/S* |
| 3 | 3.66 | 2.60 | 2.60 | 3.85 | 2.60 | N/S |
| 14 | 3.75 | 2.60 | 2.60 | 3.86 | 2.60 | N/S |
| 17 | 4.07 | 2.60 | 2.60 | 4.40 | 3.14 | N/S |
| 19 | 4.06 | 2.60 | 2.60 | 5.30 | 3.14 | N/S |
| Average | 3.96 | 2.60 | 2.60 | 4.33 | 2.85 | N/S |

*N/S = no solution

Table 16. Calculated $SN_{xeff}$ for Salem Parkway (NDT Method 1).

| Location Identification | FWD | | | Dynaflect | | |
|---|---|---|---|---|---|---|
| | BISDEF | ELSDEF | MODCOMP2 | BISDEF | ELSDEF | MODCOMP2 |
| 7 | 6.14 | 3.90 | 3.10 | 4.06 | 6.46 | 2.14 |
| 9 | 5.02 | 4.86 | 2.30 | 3.42 | 6.14 | 2.14 |
| 12 | 5.34 | 3.90 | 2.62 | 4.06 | 6.30 | 2.14 |
| 15 | 5.18 | 3.26 | 3.58 | 3.26 | 5.82 | 2.14 |
| 17 | 5.02 | 3.74 | 2.30 | 4.22 | 6.46 | 2.14 |
| Average | 5.34 | 3.93 | 2.78 | 3.80 | 6.24 | 2.14 |

Table 17. Calculated $D_{xeff}$ for Wilsonville-Hubbard Highway (NDT Method 1).

| Location Identification | FWD | | | Dynaflect | | |
|---|---|---|---|---|---|---|
| | BISDEF | ELSDEF | MODCOMP2 | BISDEF | ELSDEF | MODCOMP2 |
| 1 | 6.0 | 6.0 | N/S* | 6.0 | 7.0 | N/S |
| 2 | 6.0 | 6.0 | N/S | 6.0 | 7.0 | N/S |
| 7 | 6.0 | 6.0 | N/S | 6.0 | 7.0 | N/S |
| 13 | 6.0 | 6.0 | N/S | 6.0 | 7.0 | N/S |
| 14 | 6.0 | 6.0 | N/S | 6.0 | 7.0 | N/S |
| Average | 6.0 | 6.0 | N/S | 6.0 | 7.0 | N/S |

*N/S = no solution

Table 18. Calculated $SN_{xeff}$ for King's Valley Highway (NDT Method 2).

| Location Identification | FWD | | Dynaflect | |
|---|---|---|---|---|
| | $E_{subg}$ (psi) | $SN_{xeff}$ | $E_{subg}$ (psi) | $SN_{exff}$ |
| 3 | 8,700 | 3.61 | 28,300 | 7.87 |
| 4 | 8,900 | 3.61 | 28,300 | 7.58 |
| 5 | 9,700 | 3.41 | 26,600 | 7.26 |
| 8 | 9,600 | 3.42 | 26,600 | 8.15 |
| 18 | 11,000 | 3.31 | 32,300 | 7.59 |
| Average | 9,600 | 3.47 | 28,400 | 7.69 |

Table 19. Calculated $SN_{xeff}$ for Willamina-Salem Highway (NDT Method 2).

| Location Identification | FWD $E_{subg}$ (psi) | FWD $SN_{xeff}$ | Dynaflect $E_{subg}$ (psi) | Dynaflect $SN_{exff}$ |
|---|---|---|---|---|
| 4 | 14,600 | 3.09 | 45,300 | 7.61 |
| 7 | 16,400 | 2.83 | 54,300 | 6.89 |
| 8 | 18,100 | 2.75 | 54,300 | 6.76 |
| 12 | 19,600 | 2.95 | 67,900 | 7.39 |
| 16 | 17,300 | 2.89 | 38,800 | 7.44 |
| Average | 17,200 | 2.90 | 52,100 | 7.22 |

Table 20. Calculated $SN_{xeff}$ for Lancaster Drive (NDT Method 2).

| Location Identification | FWD $E_{subg}$ (psi) | FWD $SN_{xeff}$ | Dynaflect $E_{subg}$ (psi) | Dynaflect $SN_{exff}$ |
|---|---|---|---|---|
| 2 | 6,700 | 3.68 | 18,900 | 7.86 |
| 3 | 7,200 | 3.56 | 18,900 | 7.66 |
| 14 | 6,500 | 3.69 | 13,100 | 7.70 |
| 17 | 6,800 | 3.68 | 16,600 | 8.18 |
| 19 | 5,900 | 3.70 | 11,600 | 8.22 |
| Average | 6,600 | 3.66 | 15,800 | 7.92 |

Table 21. Calculated $SN_{xeff}$ for Salem Parkway (NDT Method 2).

| Location Identification | FWD | | Dynaflect | |
|---|---|---|---|---|
| | $E_{subg}$ (psi) | $SN_{xeff}$ | $E_{subg}$ (psi) | $SN_{exff}$ |
| 7 | 21,400 | 7.31 | 32,600 | 12.61 |
| 9 | 32,200 | 6.74 | 32,600 | 12.32 |
| 12 | 25,100 | 7.11 | 32,600 | 12.91 |
| 15 | 23,500 | 6.65 | 29,200 | 12.12 |
| 17 | 29,300 | 6.92 | 24,100 | 12.36 |
| Average | 26,300 | 6.95 | 30,200 | 12.46 |

Table 22. Calculated $D_{xeff}$ for Wilsonville-Hubbard Highway (NDT Method 2).

| Location Identification | FWD | | Dynaflect | |
|---|---|---|---|---|
| | $E_{subg}$ (psi) | $SC_{xeff}$ | $E_{subg}$ (psi) | $SC_{exff}$ |
| 1 | 7,300 | 6.0 | 10,400 | 6.0 |
| 2 | 7,800 | 6.0 | 7,400 | 6.0 |
| 7 | 6,600 | 6.0 | 7,800 | 6.0 |
| 13 | 8,400 | 6.0 | 7,500 | 6.0 |
| 14 | 6,400 | 6.0 | 7,600 | 6.0 |
| Average | 7,300 | 6.0 | 8,100 | 6.0 |

Note: NDT Method 2 is not applicable for the evaluation of rigid pavement systems. With this method, structural capacity is expressed in terms of only the PCC layer and not the other layers.

3) The NDT method 2 presents much higher $SN_{xeff}$ values than NDT method 1 using Dynaflect deflection data.

4) Maximum surface layer coefficient 0.44 was used for asphalt concrete pavements. This resulted in a relatively lower $SN_{xeff}$ for those structures with higher surface modulus and lower base layer modulus.

# 6.0 OVERLAY DESIGN

This section presents overlay designs for all five project sites using three methods: AASHTO, Caltrans, and ODOT. Each method is discussed below.

## 6.1 AASHTO Method

With the AASHTO method, overlay design was calculated based upon the existing pavement structural capacity ($SN_{xeff}$) and future traffic applications ($W_{18}$). For each project, average $SN_{xeff}$ values determined from the backcalculation results for both FWD and Dynaflect 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% reliability level (R) was chosen for the Willamina-Salem Highway and Salem Parkway projects. An 80% 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).

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 capacity for an overlaid pavement can be determined. The results of $SC_y$ for both FWD and Dynaflect on the five projects are presented in Tables 23 to 24.

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 is 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 ($R_{LY}$)

Table 23. Overlay Thickness Based on NDT Method 1.

a) FWD

| Project | $E_{sg}$* | $SC_{xeff}$* | $SC_y$ | $R_{LX}$ | $R_{LY}$ | $F_{RL}$ | $T_{AC}$ |
|---|---|---|---|---|---|---|---|
| | | <u>BISDEF</u> | | | | | |
| 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 |
| | | <u>ELSDEF</u> | | | | | |
| King's Valley Highway | 11,600 | 2.78 | 1.47 | 0.20 | 0.01 | 0.59 | 0.0 |
| Willamina-Salem Highway | 17,600 | 1.90 | 2.61 | 0.00 | 0.05 | 0.67 | 3.0 |
| Lancaster Drive | 8,600 | 2.60 | 2.90 | 0.08 | 0.07 | 0.61 | 3.0 |
| Salem Parkway | 22,700 | 3.93 | 2.58 | 0.36 | 0.05 | 0.60 | 0.5 |
| Wilsonville-Hubbard Highway | 3,900 | 6.00 | 3.90 | 1.00 | 0.13 | 1.00 | 0.0 |
| | | <u>MODCOMP2</u> | | | | | |
| King's Valley Highway | 11,600 | 2.78 | 1.47 | 0.20 | 0.01 | 0.59 | 0.0 |
| Willamina-Salem Highway | 17,600 | 1.87 | 2.61 | 0.00 | 0.05 | 0.67 | 3.1 |
| Lancaster Drive | 8,600 | 2.60 | 2.90 | 0.08 | 0.07 | 0.61 | 3.0 |
| Salem Parkway | 22,700 | 2.78 | 2.58 | 0.00 | 0.05 | 0.68 | 1.9 |
| Wilsonville-Hubbard Highway | - | N/S | - | - | - | - | - |

*Average Values

Table 23. Overlay Thickness Based on NDT Method 1 (Continued).

b) Dynaflect

| Project | $E_{sg}$* | $SC_{xeff}$* | $SC_y$ | $R_{LX}$ | $R_{LY}$ | $F_{RL}$ | $T_{AC}$ |
|---|---|---|---|---|---|---|---|
| | | BISDEF | | | | | |
| 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 | 0.58 | 0.1 |
| Wilsonville-Hubbard Highway | 8,500 | 6.00 | 2.96 | 1.00 | 0.07 | 1.00 | 0.0 |
| | | ELSDEF | | | | | |
| King's Valley Highway | 35,100 | 2.78 | 0.87 | 0.20 | 0.00 | 0.59 | 0.0 |
| Willamina-Salem Highway | 53,700 | 2.20 | 1.70 | 0.00 | 0.01 | 0.71 | 0.3 |
| Lancaster Drive | 18,000 | 2.85 | 2.21 | 0.21 | 0.03 | 0.58 | 1.2 |
| Salem Parkway | 32,300 | 6.24 | 2.26 | 1.00 | 0.03 | 1.00 | 0.0 |
| Wilsonville-Hubbard Highway | 4,600 | 7.00 | 3.69 | 1.00 | 0.12 | 1.00 | 0.0 |
| | | MODCOMP2 | | | | | |
| King's Valley Highway | 35,100 | 2.78 | 0.87 | 0.20 | 0.00 | 0.59 | 0.0 |
| Willamina-Salem Highway | 53,700 | 2.51 | 1.70 | 0.07 | 0.01 | 0.65 | 0.2 |
| Lancaster Drive | - | N/S | - | - | - | - | - |
| Salem Parkway | 32,300 | 2.14 | 2.26 | 0.00 | 0.03 | 0.69 | 1.8 |
| Wilsonville-Hubbard Highway | - | N/S | - | - | - | - | - |

*Average Values

Table 24. Overlay Thickness Based on NDT Method 2.

| Project | $E_{sg}$* | $SC_{xeff}$* | $SC_y$ | $R_{LX}$ | $R_{LY}$ | $F_{RL}$ | $T_{AC}$ |
|---|---|---|---|---|---|---|---|
| | | a) FWD | | | | | |
| King's Valley Highway | 9,600 | 3.47 | 1.59 | 0.83 | 0.01 | 0.89 | 0.0 |
| Willamina-Salem Highway | 17,200 | 2.90 | 2.63 | 0.29 | 0.05 | 0.58 | 2.1 |
| Lancaster Drive | 6,600 | 3.66 | 3.19 | 1.00 | 0.09 | 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 | 28,400 | 7.69 | 0.97 | 1.00 | 0.00 | 1.00 | 0.0 |
| Willamina-Salem Highway | 52,100 | 7.22 | 1.72 | 1.00 | 0.01 | 1.00 | 0.0 |
| Lancaster Drive | 15,800 | 7.92 | 2.32 | 1.00 | 0.04 | 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

was calculated based upon the future traffic applications and the ultimate number of repetitions that pavements fail. The failure serviceability ($P_f$) 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 overlay structural number, $SN_{OL}$, is a function of the structural capacity of the existing pavement ($SC_{xeff}$), the overlaid pavement ($SC_y$), 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 Tables 23 and 24.

## 6.2 Caltrans Method

The California Division of Highways method of overlay design for flexible pavements is based upon deflection measurements (4). In this method, the highest 80th percentile deflection value is used in the evaluation by 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 which is a function of equivalent axle load and thickness of the in-place pavement. If the tolerable deflection is greater than the representative deflection, then an overlay is not needed. If the tolerable deflection is less than the representative deflection, then the percent reduction in deflection is calculated as follows:

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

where $D_t$ = tolerable deflection.

The value of percent reduction is then used to determine the gravel equivalency factor, which is then converted to an equivalent thickness of asphalt concrete by division with a factor of 1.9.

A summary of the results for the Caltrans method for the flexible pavement sites at King's Valley Highway, Willamina-Salem Highway, Lancaster Drive, and Salem Parkway is presented in Table 25.

### 6.3 ODOT Method

The Oregon Department of Transportation employs the Caltrans deflection method with some modifications to design flexible overlays over flexible pavements (1). Deflection measurements are taken with the Dynaflect and FWD equipment and the maximum deflection values are converted to equivalent Benkelman Beam deflections.

The deflection values are adjusted for temperature condition at 70°F, averaged and then the standard deviation is determined. The highest 80th percentile deflection value is used as the design deflection. As with the Caltrans method, the design deflection is then compared with the tolerable deflection to determine if an overlay is necessary. If an overlay is required, the thickness of the overlay can be calculated from the value of percent reduction as described in the Caltrans method.

Table 25. Overlay Thickness Using Caltrans Procedure.

| Project | FWD | | | Dynaflect | | |
|---|---|---|---|---|---|---|
| | $D_{80}^{*}$ | $D_t$ | $T_{AC}$ (inches) | $D_{80}$** | $D_t$ | $T_{AC}$ (inches) |
| King's Valley Highway | 22 | 23 | 0 | 26 | 23 | 0.5 |
| Willamina-Salem Highway | 40 | 14 | 8.0 | 44 | 14 | 9.0 |
| Lancaster Drive | 28 | 16 | 3.0 | 34 | 16 | 5.5 |
| Salem Parkway | 6 | 19 | 0 | 13 | 19 | 0 |

*Deflection in mils.
**Converted to Benkelman beam value using the equation $BB = 18.33D + 0.004$

Table 26. Overlay Thickness Using ODOT Procedure.

| Project | FWD | | | Dynaflect | | |
|---|---|---|---|---|---|---|
| | $D_{80}^{*}$ | $D_t$ | $T_{AC}$ (inches) | $D_{80}$** | $D_t$ | $T_{AC}$ (inches) |
| King's Valley Highway | 22 | 23 | 0 | 18 | 23 | 0 |
| Willamina-Salem Highway | 40 | 14 | 8.0 | 42 | 14 | 5.5 |
| Lancaster Drive | 28 | 16 | 3.0 | 28 | 16 | 2.5 |
| Salem Parkway | 6 | 19 | 0 | 5 | 19 | 0 |

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

Table 26 summarizes use of the ODOT method for the four flexible pavements at the King's Valley Highway, Willamina-Salem Highway, Lancaster Drive, and Salem Parkway sites.

# 7.0 DISCUSSION OF FINDINGS

## 7.1 Backcalculation Procedures

The modulus values obtained with the three backcalculation procedures vary from each other. With BISDEF and ELSDEF, a maximum of three iterations with a tolerance of 10% were specified. The modulus range and seed modulus were selected to be the same for each test location. With MODCOMP2, a maximum of 20 iterations with a tolerance of 0.15% were used. 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. Three procedures were used, as discussed in Section 5.1.4. The use of procedure 1, (i.e., fixing the surface layer modulus which was determined from laboratory tests) resulted in predicted base and subgrade moduli which show some agreement on many of the test locations for three flexible pavements (King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive). The use of procedure 2, (i.e., fixing both surface layer modulus and subgrade modulus) resulted in similar trends (some agreement on the predicted moduli for a particular layer using the three programs) as for procedure 1. The results of using procedure 3, (i.e., fixing the subgrade modulus and solving for the surface and base layer modulus) vary for both the programs used and do not agree with the laboratory tests. However, the results from this procedure seem more reasonable for the three conventional pavement structures. These facts seem to indicate that the surface modulus is the most sensitive factor in the backcalculation methods.

## 7.2 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. Generally, the deflection data from the Dynaflect result in higher subgrade modulus. This is especially true for the NDT method 2. For the 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 important to note that deflection values generated from FWD and Dynaflect are not linearly correlated. For instance, a 9000-lb FWD load results in a deflection 19.5 mils at plate center while a 1000-lb Dynaflect load would have a 1.06 mils deformation at the same test point. The load ratio is 9, while deflection ratio is 18, twice as high as the load ratio (refer to Table 2, reading number 1). 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 has been identified. Because of the different deflection basin generated using different NDT devices under different load condition and the stress sensitivity of the pavement materials, the NDT device does have considerable influence on the backcalculated moduli which, in turn, affects the resulting overlay design thickness.

## 7.3 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. The background of these two methods are different. 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. The two methods can result in different solutions.

## 7.4 Overlay Design Methods

The overlay thicknesses determined from the three design methods, AASHTO, Caltrans, and ODOT, are summarized in Table 27. As can be seen from the table, the King's Valley Highway and Salem Parkway projects seem to have no need of an overlay as calculated using all three methods. With the exception of NDT method 2 using Dynaflect data, all the procedures demonstrate 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 a thickness in excess of 5.5 in. is required by the Caltrans and ODOT methods. For the Lancaster Drive site, no overlay is required using both NDT methods 1 and 2, while results from both Caltrans and ODOT methods show an overlay is required, the thickness varies from 2.5 in. to 5.5 in. Since Lancaster Drive had been overlaid the previous year (1986), it would seem that the AASHTO procedure provided the most 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.

Preliminary analysis of these results seems to lead to either one of the following conclusions: the Caltrans and ODOT methods provide an overdesign of the overlay thickness and the AASHTO method(s) provide a more reasonable result; or the Caltrans and ODOT methods provide the reasonable design and the AASHTO method(s) provide an underdesign. Experience and the existing pavement condition would indicate the latter to be true.

Table 27. Comparison of Three Overlay Design Methods.

| Project Site | AASHTO | | | | Caltrans | | ODOT | |
|---|---|---|---|---|---|---|---|---|
| | NDT Method 1 | | NDT Method 2 | | | | | |
| | FWD | Dynaflect | FWD | Dynaflect | FWD | Dynaflect | FWD | Dynaflect |
| King's Valley* Highway | 0 | 0 | 0 | 0 | 0 | 0.5 | 0 | 0 |
| Willamina-Salem Highway | 3.4** | 1.0 | 2.1 | 0 | 8.0 | 9.0 | 8.0 | 5.5 |
| Lancaster Drive | 0 | 0 | 0 | 0 | 3.0 | 5.5 | 3.0 | 2.5 |
| Salem Parkway | 0 | 0.1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Wilsonville-Hubbard Highway*** | 0 | 0 | 0 | 0 | N/A | | N/A | |

*This pavement would probably require a chip seal to prevent water infiltration. Its structural capacity is good.

** Value based on BISDEF results.

***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.

# 8.0 CONCLUSIONS AND RECOMMENDATIONS

## 8.1 Conclusions

Preliminary conclusions can be made from the data analyzed thus far.

1) The AASHTO method is based on the concept of reliability of design as well as the remaining life of the pavement; the Caltrans and ODOT methods are based on the highest 80th percentile deflection value while the remaining life of the pavement is ignored.
2) NDT method 1 still needs further research to develop and test a reliable backcalculation method. The study in this report shows that the three backcalculation programs seem to work relatively well with those conventional pavement structures.
3) NDT method 2 can be utilized with reasonable confidence when subgrade material properties are well determined.
4) The deflection data collected using FWD or Dynaflect can result in different overlay design if there exists poor correlation between FWD and Dynaflect deflection data.

## 8.2 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 in verifying the backcalculated values. In some cases, the laboratory results should be used in the programs as a fixed input to determine the moduli of the other layers.

2) Considerable judgment must be made in determining the layer coefficient as the relationships between moduli and layer coefficients are not well defined.

3) The differences in calculated overlay thickness between the AASHTO and ODOT procedures should be resolved.

4) A comparison of the three design procedures (AASHTO, Caltrans, and ODOT) reveals significant differences between the calculated overlay designs. Until these differences are resolved, the 1986 AASHTO procedure should be used with caution.

## 8.3 Recommended Research

Based upon the evaluation presented in this report, the following research seems necessary:

1. Validity of backcalculated moduli as compared to laboratory test results.
2. Correlation of using various NDT devices for pavement evaluation.
3. Mechanistic approach for overlay design rather than empirical method.

## 9.0 REFERENCES


1. Oregon State Highway Division, "Flexible Pavement Design Procedure," 1956.

2. American Association of State Highway and Transportation Officials, "AASHTO Interim Guide for Design of Pavement Structures," 1981.

3. Portland Cement Association, "Thickness Design for Concrete Highway and Street Pavements," 1984.

4. California Department of Transportation, "Asphalt Concrete Overlay Design Manual," 1979.

5. American Association of State Highway and Transportation Officials, "AASHTO Guide for Design of Pavement Structures," 1986.

6. Tholen, O., "Falling Weight Deflectometer, A Device for Bearing Capacity Measurements; Properties and Performance," Department of Highway Engineering, Royal Institute of Technology, Stockholm, Sweden, 1980.

7. Tholen, O., Sharma, J., and Terrel, R., "Comparison of Falling Weight Deflectometer with Other Deflection Testing Devices," TRR 1007, Washington, D.C., 1985.

8. Bush, A.J. and Alexander, D.R., "Pavement Evaluation Using Deflection Basin Measurement and Layered Theory," TRR 1022, TRB, Washington, D.C., 1985.

9. Bush, A.J., III, "Nondestructive Testing for Light Aircraft Pavements, Phase II, Development of Nondestructive Evaluation Methodology," Final Report No. FAA-RD-80-9-II, U.S. DOT, November 1980.

10. Lytton, R.L., Roberts, F.L., and Stoffels, S., "Determination of Asphaltic Concrete Pavement Structural Properties by Nondestructive Testing," Final Report, NCHRP, Project 10-27.

11. Irwin, L.H., "User's Guide to MODCOMP2, Version 2.1," Cornell University, Local Roads Program, Ithaca, NY, November 1983.

12. Rwebangira, Theo et al., "Sensitivity Analysis of Selected Backcalculation Procedures," paper presented at Annual Meeting of Transportation Research Board, January 1987.

13. Hicks, R.G. and McHattie, R.L., "Use of Layered Theory in the Design and Evaluation of Pavement Systems," Report No. FHWA-AK-RD-83-8, July 1982.

14. Zhou, Haiping et al., "Development of an Improved Overlay Design Procedure for Oregon: Vol. III--Field Manual, FHWA-OR-RD-87-XX, Federal Highway Administration, Nov. 1987.

APPENDIX A

PHOTOS OF PAVEMENT CONDITION

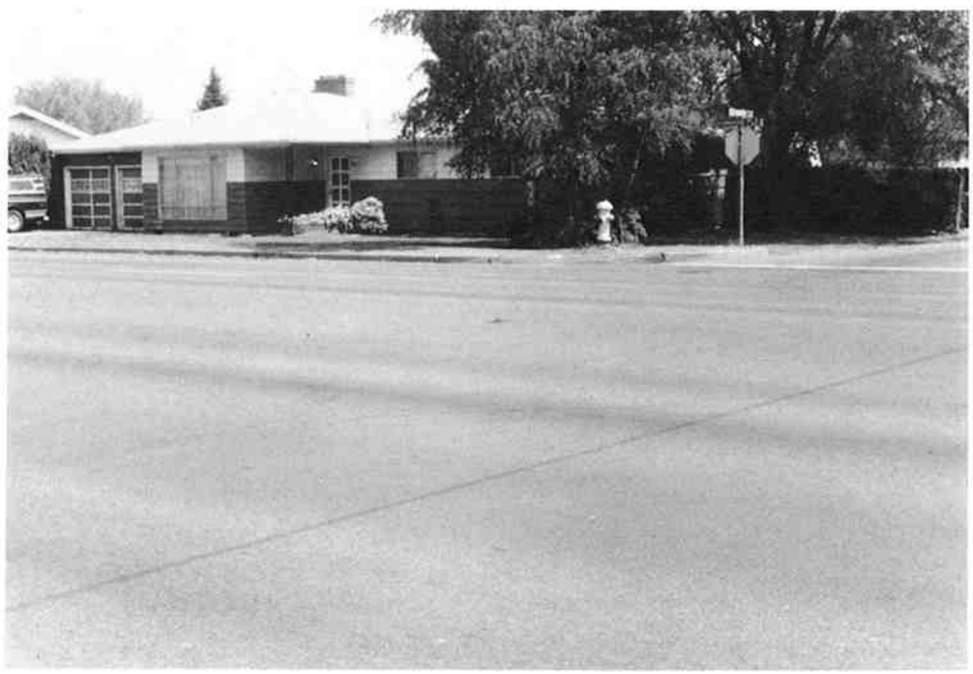
April 1987

a) Overall View

b) Evidence of Cracking

Figure A.1. Photos of King's Valley Highway.

a) Looking South



b) Looking Across the Roadway

Figure A.2. Photos of Lancaster Drive.

a) Overall View

b) Closeup of Cracking

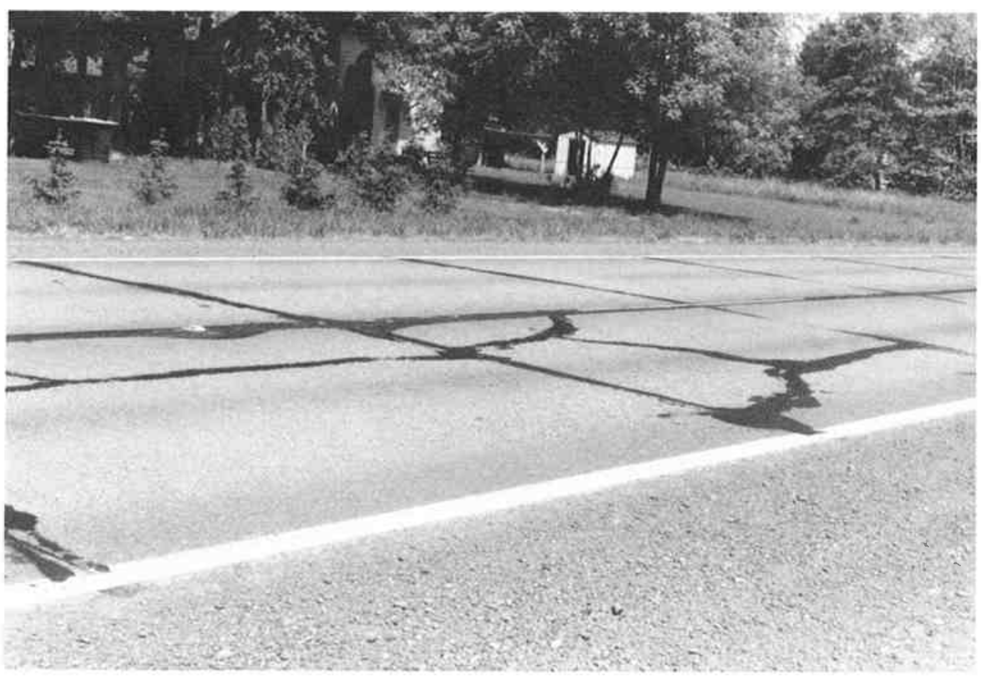
Figure A.3. Willamina-Salem Highway.

a) Overview View

b) Looking Across Roadway

Figure A.4. Photos of Salem Parkway.

a) Overall View



b) Closeup of Pavement Distress

Figure A.5. Photos of Wilsonville-Hubbard Highway.

# APPENDIX B

# CALCULATION OF STRUCTURAL CAPACITY FOR EXISTING PAVEMENTS

## APPENDIX B

## CALCULATION OF STRUCTURAL CAPACITY FOR EXISTING PAVEMENTS

This section presents the calculation of structural capacity for existing pavements using both NDT method 1 and method 2.

### B.1 Calculation of $SC_{xeff}$ Using NDT Method 1

Determination of $SC_{xeff}$ using NDT method 1 is based upon the moduli backcalculated from measured deflection basin data. The moduli are used to determine layer coefficient ($a_i$) corresponding to each pavement layer using various charts given in AASHTO Guide. The $SN_{xeff}$ may then be calculated using the equation $SN = \Sigma\ a_i h_i$, where $h_i$ is the thickness for ith layer. The procedure is simple; however, if the backcalculated moduli are not in the range of those charts used for determining the layer coefficient, engineering judgment would have to be made to determine reasonable layer coefficient.

Not all backcalculated results are used for later analysis. For King's Valley Highway, Willamina-Salem Highway, and Lancaster Drive projects, backcalculation results from procedure 3 are selected for determining the $SN_{xeff}$ and, consequently, the $SN_{xeff}$ thus determined are used for overlay design purposes. For Salem Parkway and Wilsonville-Hubbard Highway projects, results from procedure 1 are selected.

Calculation of the structural capacity for five projects are tabulated and presented on the following pages.

Table B1. Calculation of $SN_{xeff}$ for King's Valley Highway (NDT Method 1).

| Location Identification | BISDEF Surface* | BISDEF Base | BISDEF $SN_{xeff}$ | ELSDEF Surface | ELSDEF Base | ELSDEF $SN_{xeff}$ | MODCOMP2 Surface | MODCOMP2 Base | MODCOMP2 $SN_{xeff}$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | | a) FWD | | | | | |
| 3 | 0.44 | 0.12 | 4.32 | 0.44 | 0.01** | 2.78 | 0.44 | 0.01 | 2.78 |
| 4 | 0.44 | 0.11 | 4.18 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 5 | 0.43 | 0.08 | 3.70 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 8 | 0.43 | 0.08 | 3.70 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 18 | 0.41 | 0.06 | 3.30 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| | | | | b) Dynaflect | | | | | |
| 3 | 0.44 | 0.04 | 3.20 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 4 | 0.44 | 0.02 | 2.92 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 5 | 0.44 | 0.02 | 2.92 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 8 | 0.44 | 0.06 | 3.48 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |
| 18 | 0.44 | 0.05 | 3.34 | 0.44 | 0.01 | 2.78 | 0.44 | 0.01 | 2.78 |

*Maximum layer coefficient 0.44 is used for surfacing layer.
**Assumed value for modulus less than 9,000 psi.

Table B2. Calculation of $SN_{xeff}$ for Willamina-Salem Highway (NDT Method 1).

| Location Identification | BISDEF | | | ELSDEF | | | MODCOMP2 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Surface* | Base | $SN_{xeff}$ | Surface | Base | $SN_{xeff}$ | Surface | Base | $SN_{xeff}$ |
| | | | | a) FWD | | | | | |
| 4 | 0.30 | 0.01 | 1.77 | 0.35 | 0.01** | 2.04 | 0.33 | 0.01 | 1.93 |
| 7 | 0.27 | 0.01 | 1.61 | 0.31 | 0.01 | 1.82 | 0.31 | 0.01 | 1.82 |
| 8 | 0.27 | 0.01 | 1.61 | 0.31 | 0.01 | 1.82 | 0.31 | 0.01 | 1.82 |
| 12 | 0.30 | 0.01 | 1.77 | 0.34 | 0.01 | 1.98 | 0.32 | 0.01 | 1.88 |
| 16 | 0.27 | 0.01 | 1.61 | 0.31 | 0.01 | 1.82 | 0.32 | 0.01 | 1.88 |
| | | | | b) Dynaflect | | | | | |
| 4 | 0.30 | 0.02 | 1.95 | 0.41 | 0.01 | 2.35 | 0.44 | 0.01 | 2.51 |
| 7 | 0.25 | 0.01 | 1.51 | 0.36 | 0.01 | 2.09 | 0.44 | 0.01 | 2.51 |
| 8 | 0.22 | 0.01 | 1.35 | 0.34 | 0.01 | 1.98 | 0.44 | 0.01 | 2.51 |
| 12 | 0.35 | 0.01 | 2.04 | 0.39 | 0.01 | 2.25 | 0.44 | 0.01 | 2.51 |
| 16 | 0.29 | 0.02 | 1.90 | 0.41 | 0.01 | 2.35 | 0.44 | 0.01 | 2.51 |

*Maximum layer coefficient 0.44 is used for surfacing layer.
**Assumed value for modulus less than 9,000 psi.

Table B3. Calculation of $SN_{xeff}$ for Lancaster Drive (NDT Method 1).

| Location Identification | BISDEF | | | ELSDEF | | | MODCOMP2 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Surface* | Base | $SN_{xeff}$ | Surface | Base | $SN_{xeff}$ | Surface | Base | $SN_{xeff}$ |
| | | | | a) FWD | | | | | |
| 2 | 0.35 | 0.13 | 4.27 | 0.44 | 0.01** | 2.60 | 0.44 | 0.01 | 2.60 |
| 3 | 0.37 | 0.09 | 3.66 | 0.44 | 0.01 | 2.60 | 0.44 | 0.01 | 2.60 |
| 14 | 0.42 | 0.08 | 3.75 | 0.44 | 0.01 | 2.60 | 0.44 | 0.01 | 2.60 |
| 17 | 0.38 | 0.11 | 4.07 | 0.44 | 0.01 | 2.60 | 0.44 | 0.01 | 2.60 |
| 19 | 0.41 | 0.10 | 4.06 | 0.44 | 0.01 | 2.60 | 0.44 | 0.01 | 2.60 |
| | | | | b) Dynaflect | | | | | |
| 2 | 0.41 | 0.11 | 4.24 | 0.44 | 0.02 | 2.78 | N/S | | |
| 3 | 0.34 | 0.11 | 3.85 | 0.44 | 0.01 | 2.60 | N/S | | |
| 14 | 0.44 | 0.08 | 3.86 | 0.44 | 0.01 | 2.60 | N/S | | |
| 17 | 0.44 | 0.11 | 4.40 | 0.44 | 0.04 | 3.14 | N/S | | |
| 19 | 0.44 | 0.16 | 5.30 | 0.44 | 0.04 | 3.14 | N/S | | |

*Maximum layer coefficient 0.44 is used for surfacing layer.
**Assumed value for modulus less than 9,000 psi.

Table B4. Calculation of $SN_{xeff}$ for Salem Parkway (NDT Method 1).

| Location Identification | BISDEF Surface | BISDEF Base | BISDEF $SN_{xeff}$ | ELSDEF Surface | ELSDEF Base | ELSDEF $SN_{xeff}$ | MODCOMP2 Surface | MODCOMP2 Base | MODCOMP2 $SN_{xeff}$ |
|---|---|---|---|---|---|---|---|---|---|
| | | | | a) FWD | | | | | |
| 7 | 0.44 | 0.26 | 6.14 | 0.44 | 0.12 | 3.90 | 0.44 | 0.07 | 3.10 |
| 9 | 0.44 | 0.19 | 5.02 | 0.44 | 0.18 | 4.86 | 0.44 | 0.02 | 2.30 |
| 12 | 0.44 | 0.21 | 5.34 | 0.44 | 0.12 | 3.90 | 0.44 | 0.04 | 2.62 |
| 15 | 0.44 | 0.20 | 5.18 | 0.44 | 0.08 | 3.26 | 0.44 | 0.10 | 3.58 |
| 17 | 0.44 | 0.19 | 5.02 | 0.44 | 0.11 | 3.74 | 0.44 | 0.02 | 2.30 |
| | | | | b) Dynaflect | | | | | |
| 7 | 0.44 | 0.13 | 4.06 | 0.44 | 0.28 | 6.46 | 0.44 | 0.01 | 2.14 |
| 9 | 0.44 | 0.09 | 3.42 | 0.44 | 0.26 | 6.14 | 0.44 | 0.01 | 2.14 |
| 12 | 0.44 | 0.13 | 4.06 | 0.44 | 0.27 | 6.30 | 0.44 | 0.01 | 2.14 |
| 15 | 0.44 | 0.08 | 3.26 | 0.44 | 0.24 | 5.82 | 0.44 | 0.01 | 2.14 |
| 17 | 0.44 | 0.14 | 4.22 | 0.44 | 0.28 | 6.46 | 0.44 | 0.01 | 2.14 |

Table B5. Calculation of $SC_{xeff}$ for Wilsonville-Hubbard Highway (NDT Method 1).

| Location Identification | BISDEF | | | ELSDEF | | | MODCOMP2 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Surface* | Base | $SN_{xeff}$** | Surface | Base | $SN_{xeff}$ | Surface | Base | $SN_{xeff}$ |
| | | | | a) FWD | | | | | |
| 1 | 7.5 | 0 | 6.0 | 7.5 | 0 | 6.0 | N/S | | |
| 2 | 7.5 | 0 | 6.0 | 7.5 | 0 | 6.0 | N/S | | |
| 7 | 7.5 | 0 | 6.0 | 7.5 | 0 | 6.0 | N/S | | |
| 13 | 7.5 | 0 | 6.0 | 7.5 | 0 | 6.0 | N/S | | |
| 14 | 7.5 | 0 | 6.0 | 7.5 | 0 | 6.0 | N/S | | |
| | | | | b) Dynaflect | | | | | |
| 1 | 7.5 | 0 | 6.0 | 7.5 | 0.20 | 7.0 | N/S | | |
| 2 | 7.5 | 0 | 6.0 | 7.5 | 0.20 | 7.0 | N/S | | |
| 7 | 7.5 | 0 | 6.0 | 7.5 | 0.20 | 7.0 | N/S | | |
| 13 | 7.5 | 0 | 6.0 | 7.5 | 0.20 | 7.0 | N/S | | |
| 14 | 7.5 | 0 | 6.0 | 7.5 | 0.20 | 7.0 | N/S | | |

*$D_{xeff}$ determined using modulus value of 4,977,000 psi.

**$SC_{xeff} = 0.8\ D_{xeff} + SN_{xeff-rp}$.

## B2. Calculation of SCxeff Using NDT Method 2

Calculations of $SC_{xeff}$ are conducted using a computer program developed by the authors. Description of this program and necessary background for using NDT method 2 may be found in Volume III of this report. The following presents an example showing the procedures of calculating the $SC_{xeff}$.

Deflection data are selected from Table 3, reading number 4, which gives maximum deflection 36.5 (mils) at center of FWD and 4.47 (mils) at fourth sensor. A temperature adjustment factor 1.03 is applied resulting in a maximum deflection of 37.67 (mils).

Other data are given as follows:

1) Existing pavement structure

   Surface: 5.3 in. asphalt concrete

   Base: 18.0 in. granular base

2) Material properties

   Surface: estimated modulus 200,000 psi, Poisson's ratio 0.35

   Base: estimated modulus 6,000 psi, Poisson's ratio 0.40

   Subgrade: estimated modulus 15,000 psi, Poisson's ratio 0.50

3) Initial design layer coefficients:

   Surface: 0.44

   Base: 0.07

4) NDT device:

   Equipment: FWD

   Dynamic load: 9,000 lbs

   Load plate radius ($a_c$): 5.9 in.

The following steps are used to determine the $SC_{xeff}$:

1. Compute the $H_e$ value for the pavement

$$H_e = 0.9 \sum_{i=1} h_i[[E_i(1 - u_{sg}^2)]/[E_{sg}(1 - u_i^2)]^{1/3}$$

$$= 0.9 \sum_{i=1} 5.3[[200{,}000(1 - 0.5^2)]/[15{,}000(1 - 0.35^2)]^{1/3}$$

$$+ 18[[6{,}000(1 - 0.5^2)]/[15{,}000(1 - 0.40^2)]^{1/3}$$

$$= 0.9\ \Sigma(11.93 + 12.77) = 22.23$$

2. Compute the $H_e/a_c$ ratio

$$H_e/a_c = 22.23/5.9 = 3.77$$

3. Determine the $F_b$ value

$$F_b = 0.26$$

4. Determine $a_e$

$$a_e = a_c/F_b = 5.9/0.26 = 22.69$$

5. Compute the $r/a_e$ ratio

$$r/a_e = 36/22.69 = 1.59 > 1$$

6. Compute the subgrade modulus

$$E_{sg} = PS_f/(rd_r)$$

$$= 9{,}000*0.2686/(36*4.47*1.03)$$

$$= 14{,}585 \text{ psi}$$

7. Compute $H_e/a_c$ ratio using the following equation:

$$\frac{h_e}{a_c} = \frac{209.3\ SN}{a_c} \left[\frac{(1-u_{sg}^2)}{E_{sg}}\right]^{1/3}$$

Assume SN = 2,

$$\frac{h_e}{a_c} = \frac{209.3*2}{5.9} \left[\frac{(1-0.5^2)}{14{,}585}\right]^{1/3} = 2.638$$

8. Compute $F_b$ value using the following equation:

$$F_b = \left[\left[1 + \left[\frac{h_e}{a_c}\right]^2\right]^{0.5} - \frac{h_e}{a_c}\right] \left[1 + \frac{(h_e/a_c)}{2(1-u_{sg})[1 + (h_e/a_c)^2]^{0.5}}\right]$$

$$= \left[\left[1 + 2.638^2\right]^{0.5} - 2.638\right] \left[1 + \frac{2.638}{2(1 - 0.5)[1 + 2.638^2]^{0.5}}\right]$$

$$= 0.183*1.935 = 0.354$$

9. Compute deflection $d_o$ using the following equation:

$$d_o = \left[\frac{2P(0.0043*h_t)^3}{3.1416\ a_c\ SN^3}\right] \left[1 + F_b\left[\frac{SN^3(1-u_{sg}^2)}{E_{sg}(0.0043*h_t)^3} - 1\right]\right]$$

$$= \left[\frac{2*9{,}000(0.0043*23.3)^3}{3.1416*5.9*2^3}\right] \left[1 + 0.354\left[\frac{2^3(1-0.5^2)}{14{,}585(0.0043*23.3)^3} - 1\right]\right]$$

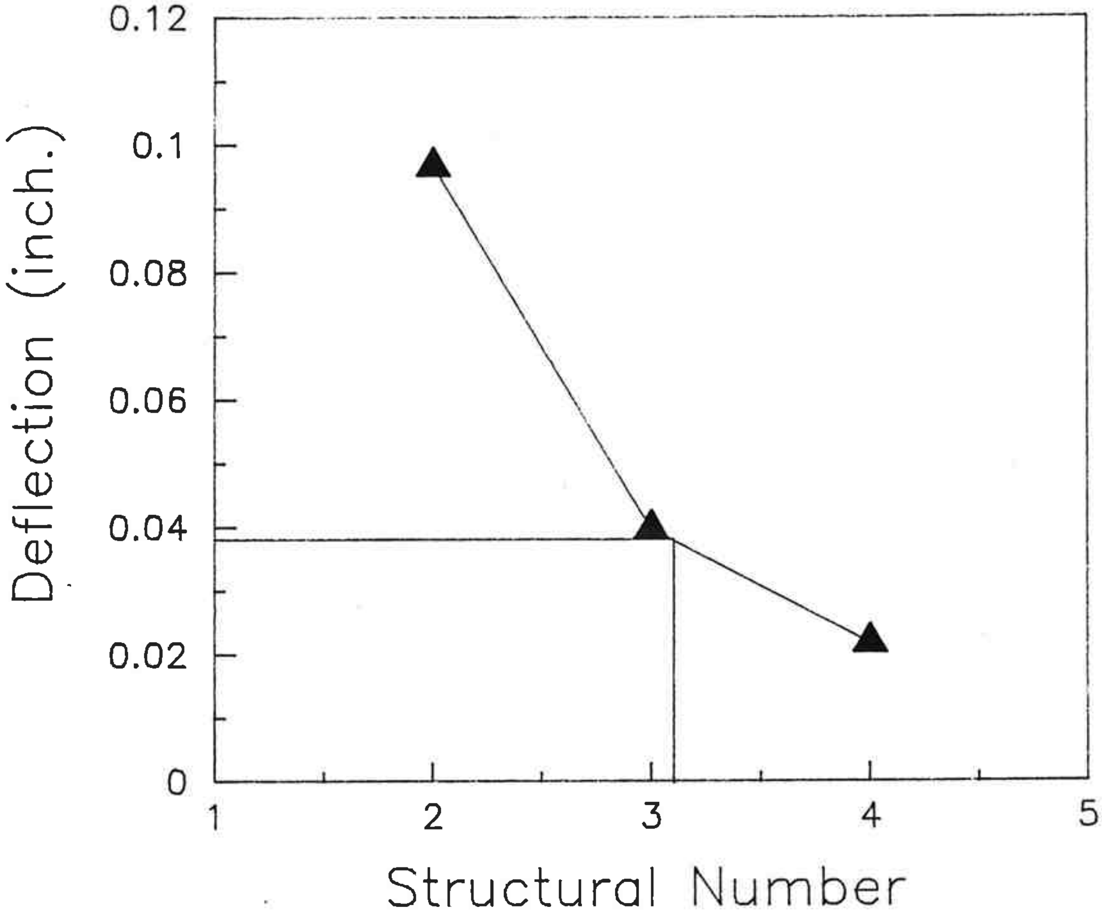
$$= 0.122*0.791 = 0.0965 \text{ in.}$$

10. Repeat procedures 7 to 9 to obtain $d_o$ values at varying SN. The calculated $d_o$'s are listed in the following table

| Assumed SN | Fb Value | Predicted $d_o$ |
|---|---|---|
| SN = 2 | 0.354 | 0.0965 in. |
| SN = 3 | 0.242 | 0.0395 in. |
| SN = 4 | 0.186 | 0.0217 in. |

In order to determine the estimated $SN_{xeff}$, a plot of assumed SN versus predicted $d_o$ values can be made as shown below, and the corresponding $SN_{xeff}$

value interpolated for $d_o$ = 36.67 mils (adjusted). This results in an $SN_{xeff}$ = 3.07.




The above steps are used for the determination of $SC_{xeff}$ for other test locations.