DEVELOPMENT OF AN IMPROVED OVERLAY DESIGN PROCEDURE FOR OREGON

VOLUME II - EVALUATION OF PROCEDURE

by

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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 fourth step 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

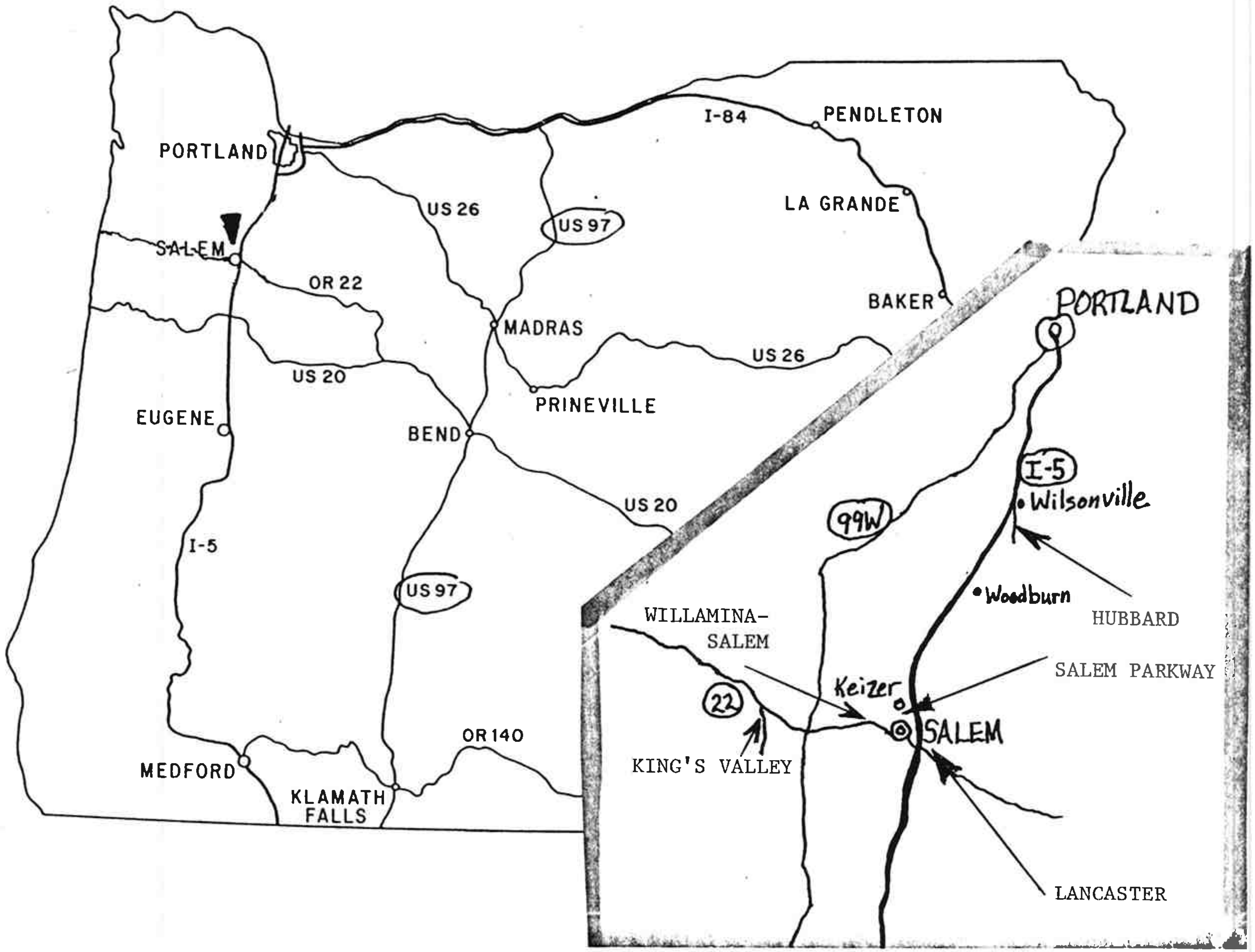


Figure 1. Location Map of the Project Sites.

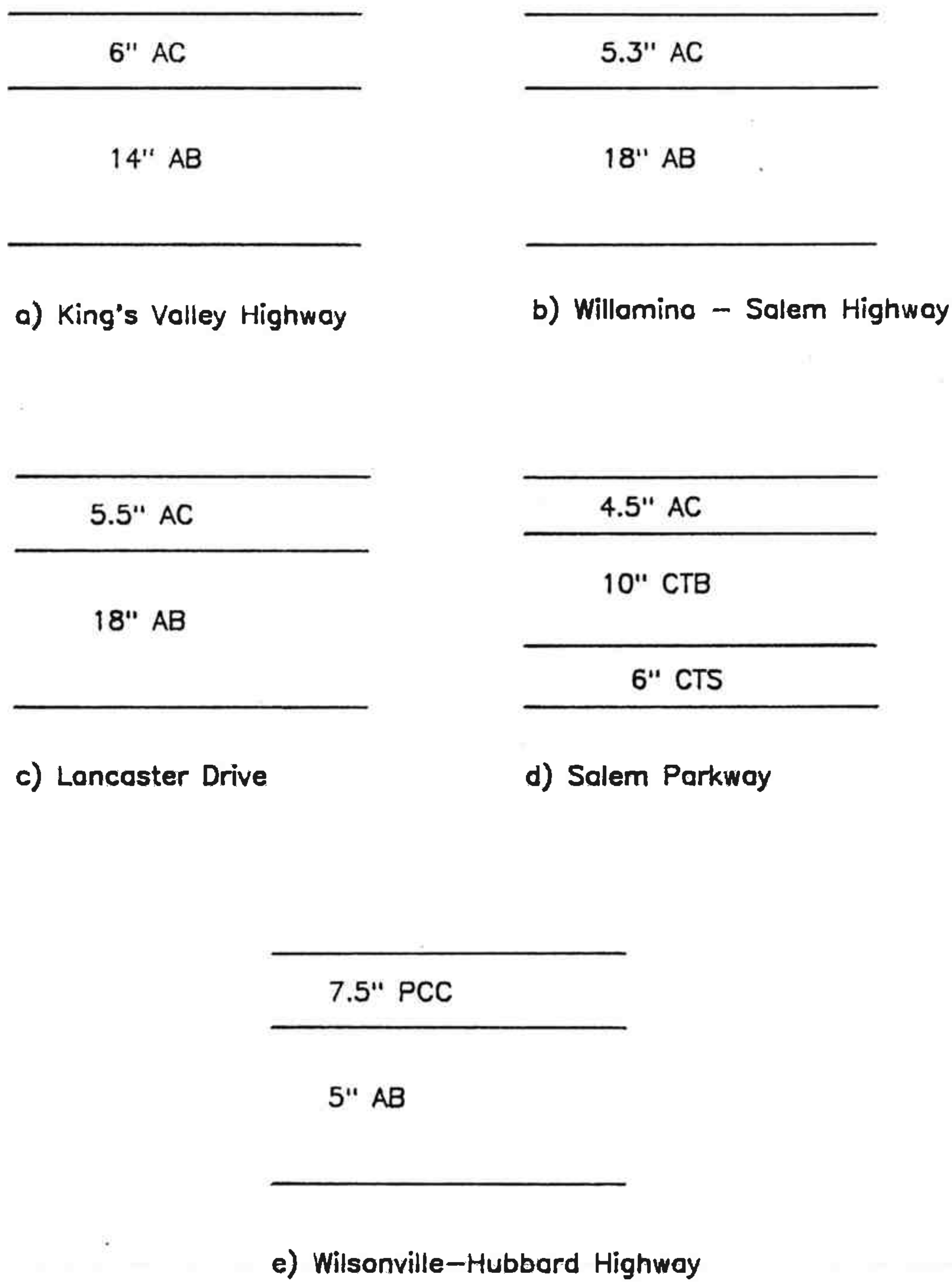


Figure 2. Cross Sections of Pavements Analyzed.

Table 1. Summary of Project Data.

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

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

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

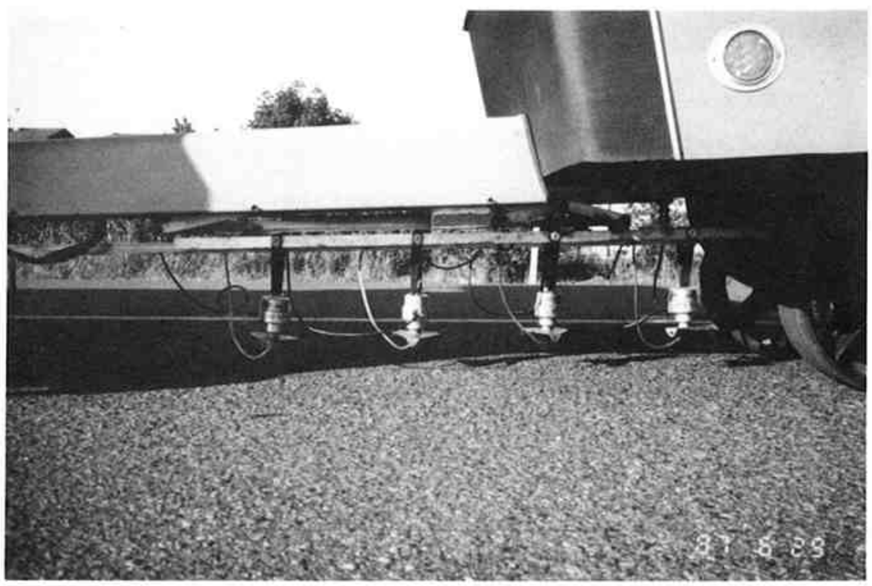
Pavement surface deflection data were measured at 50-ft intervals for 1000-ft sections for each project. The measurements were taken with the KUAB Falling Weight Deflectometer (FWD) and the Dynaflect, both owned and operated by ODOT. For each site, deflection basin measurements were taken at 50-ft intervals 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

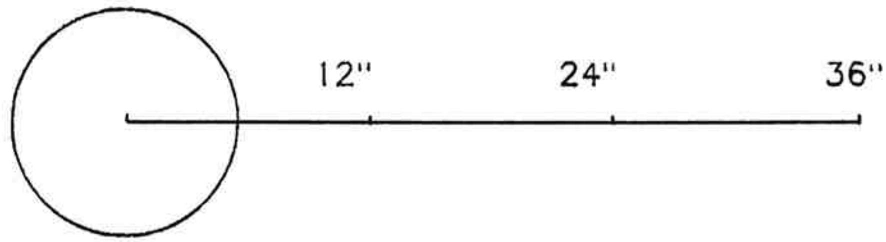
b) Closeup of Internal Working System

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

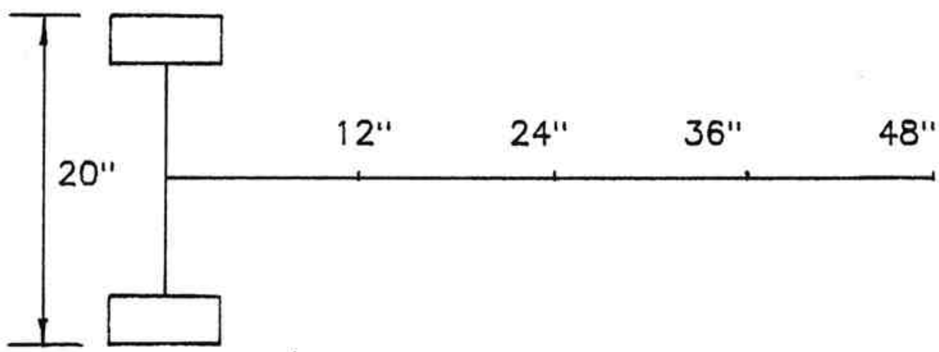
weights from different heights. By varying the drop height, the load at the pavement surface was varied from 4900 to 11,300 lbs. The two-mass system is used to create a smooth load pulse similar to that created by a moving wheel load (6,7). Surface deflections were measured with four seismic transducers (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.



a) FWD



b) Dynaflect

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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