DEVELOPMENT OF AN IMPROVED OVERLAY DESIGN PROCEDURE FOR OREGON

Vol: I - Interim Report

Background and Framework

by

Constance L. Eichhorn
Research Assistant
Oregon State University

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

Ronald Noble
Materials Unit Engineer
Oregon State Highway Division

and

Haiping Zhou
Research Assistant
Oregon State University

for

Oregon Department of Transportation

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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 (31). The Portland Cement Association (PCA) and American Association of State Highway and Transportation Officials (AASHTO) method are employed for portland cement overlays (1,38). The Dynaflect is used to obtain deflections for the flexible overlay design procedure. Presently, the maximum deflection obtained by the Dynaflect equipment is converted to an equivalent Benkelman Beam deflection and then used in the modified Caltrans method (9). For portland cement concrete overlays, the PCA and AASHTO methods often give inconsistent values.

In both instances, the generated data are insufficient to define accurately the structural adequacy of the existing pavement. In addition, the current procedures do not accurately 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 a more efficient overlay design, a new overlay design method is needed. The development and subsequent use of this new procedure should increase the reliability in determining the remaining life, while decreasing maintenance expenditures from unexpected rehabilitation costs.

1.2 Objectives

The overall objective of this study was to develop an improved overlay design procedure for both flexible and rigid overlays. Specific objectives include the following:

1) Review of current practices (ODOT and others) for evaluating overlay design needs for existing pavements,
2) Evaluate deflection equipment for use in an improved overlay design procedure,
3) Develop a framework to evaluate overlay design needs,
4) Test the use of the procedure on selected projects, and
5) Implement its use in the State of Oregon.

This report presents the results of the first year's effort to accomplish the stated objectives.

1.3 Study Approach

The approach used to satisfy the project objectives is given in Figure 1.1. Basically, the approach consisted of completing the following steps:

1) Collect and review existing overlay design procedures which are summarized in Appendix A.
2) Current overlay design procedures employed in Oregon were summarized and the proposed AASHTO procedure evaluated.
3) Develop an improved framework for an overlay design method. This includes evaluation of deflection equipment, traffic and layer moduli determination.

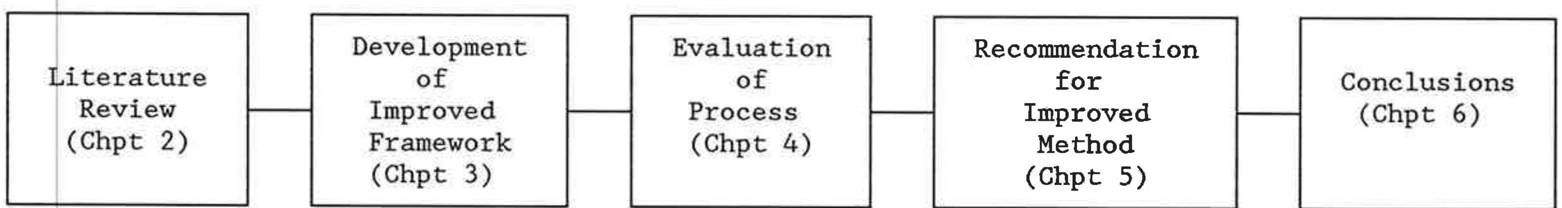


Figure 1.1. Study Approach.

4) Evaluate the developed methodology on selected projects. The proposed design results are compared with those currently obtained with existing methods.

5) Develop plans for implementation of the improved method in Oregon.

6) Summarize and document all conclusions and recommendations for this effort.

2.0 LITERATURE REVIEW

The development of an improved overlay design procedure for Oregon highways includes the analysis of current procedures and a review of other methods available. A Transportation Research Information Service (TRIS) literature search was performed to gather the necessary information to formulate an appropriate overlay design procedure. Many deflection-based methods, as well as mechanistic methods, were reviewed to determine their suitability for use in Oregon (see Appendix A). Since ODOT uses the Dynaflect, and has recently purchased a Falling Weight Deflectometer (FWD), overlay design methods which use the deflection basin technique could be employed.

The proposed AASHTO method (2) which uses deflection basin techniques and considers the remaining life of the pavement is also evaluated in this chapter. Since it considers the contribution of each layer of the pavement structure, determination of a more accurate in situ condition of the pavement may be possible.

2.1. Oregon Current Practices

2.1.1 Flexible Overlays for Flexible Pavements

Currently, the Oregon Department of Transportation employs the Caltrans deflection method (9) with some modifications to design flexible overlays over flexible pavements. Deflection measurements are taken with the Dynaflect equipment and converted to equivalent maximum Benkelman Beam deflections using Figure 2.1. The exponential curve is usually used. Tests are taken in a random 1000-ft (304.8 m) section for every mile of the intended overlay project. Twenty-one tests, one performed every 50 ft (15.24 m), comprise the database for every mile of roadway. The 21 deflection values are adjusted for

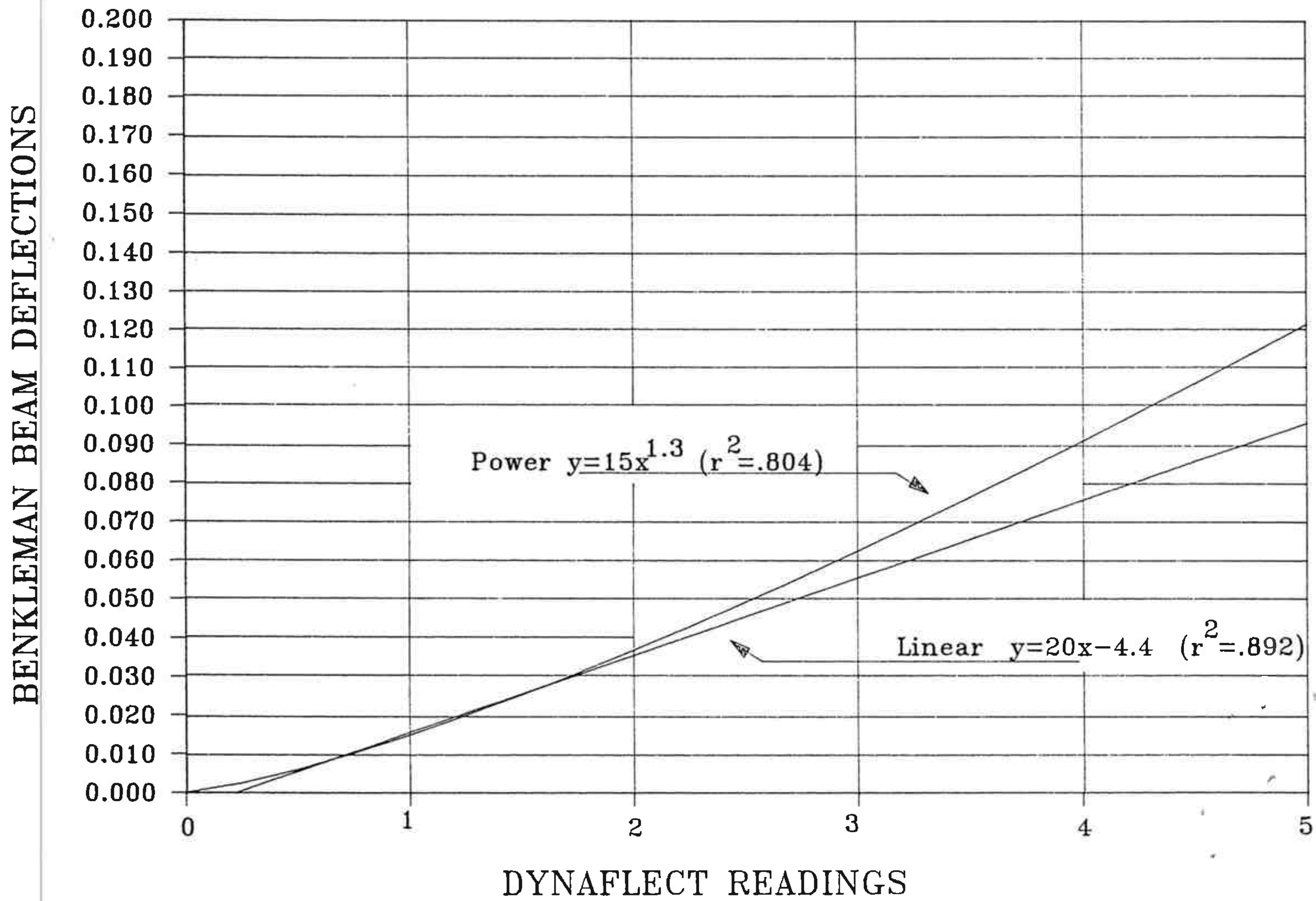


Figure 2.1. Dynaflect Reading vs. Benkelman Beam Deflections for ODOT (31).

temperature conditions (Figure 2.2), averaged, and then the standard deviation is determined. The 80th percentile value (mean +.84s) is used as the design deflection.

The tolerable deflection is a function of the traffic loading and the thickness of the in-place pavement as shown in Figure 2.3. If the design deflection is less than the tolerable deflection then no overlay is required. If the design deflection is greater than the tolerable deflection then the percent reduction deflection is calculated. This value is entered into Figure 2.4 and the equivalent crushed base value is read off the horizontal axis. This value is converted to an asphalt concrete thickness by dividing by the crushed base equivalency factor (which is 2.0). The values for each mile are noted, and the worst case value is used to determine the overlay thickness for the entire project. If one particular area is severely distressed, it may be treated separately. Advantages and disadvantages of this method are given in Table 2.1.

2.1.2 PCC Overlays for Flexible Pavements

Recently, many portland cement overlays have been placed over flexible pavements in Oregon. For these situations ODOT uses both the PCA method (38) and the AASHTO method (1). The required thickness is calculated for both procedures and then engineering judgment is used to determine the design thickness.

PCA Method (38). The procedure for determining overlay thickness over flexible pavements is essentially the same procedure as that for new PCC pavements. Design factors include:

1) type of joint and shoulder,

2) concrete flexural strength (28 day),

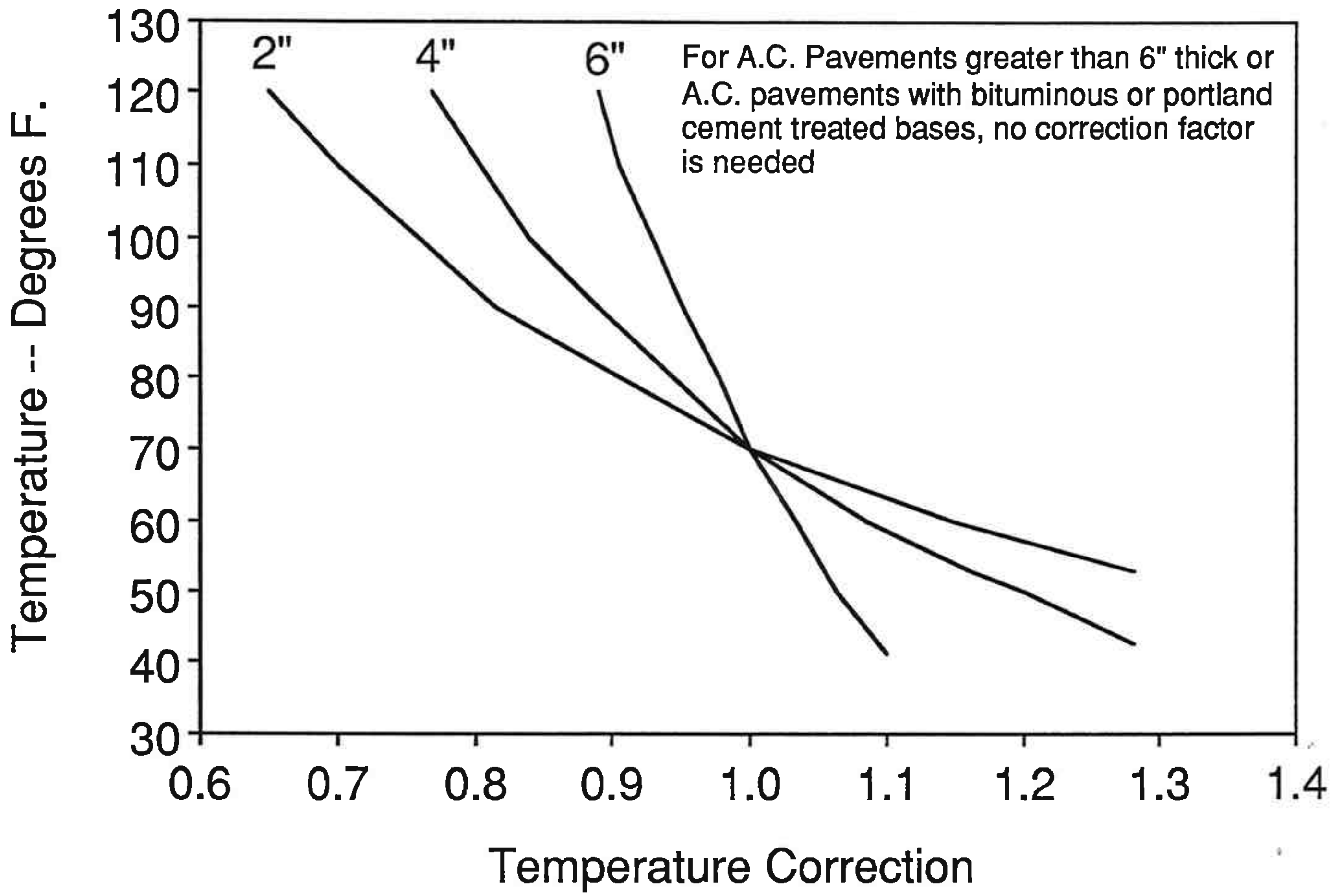
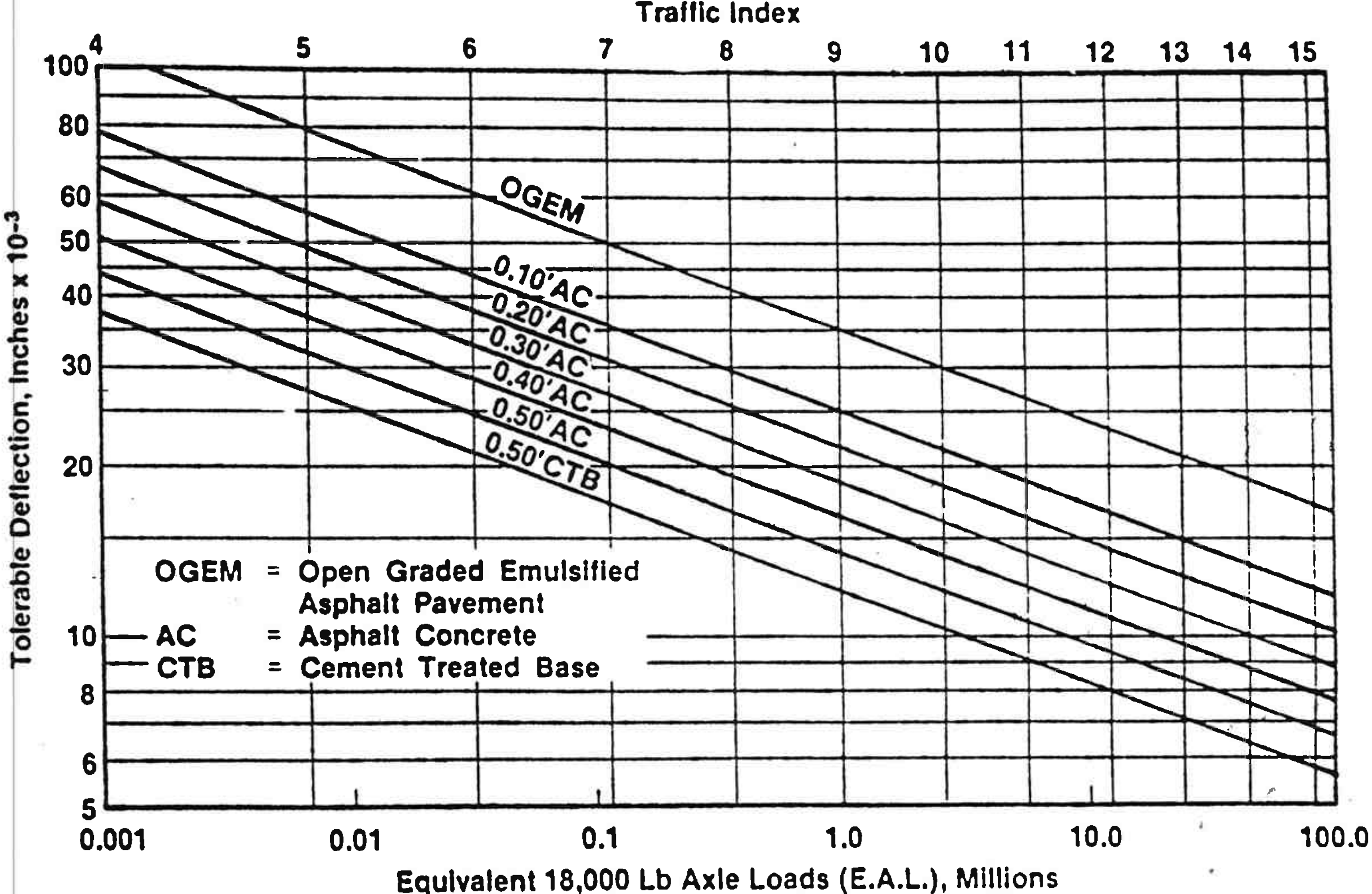


Figure 2.2. Temperature Correction Factor for Various Asphalt Pavement Thicknesses (27).

TOLERABLE DEFLECTION CHART - OREGON DEPARTMENT OF TRANSPORTATION (8)



1 In. = 2.54 cm
1 Lb = 4.45 N

Figure 2.3. Tolerable Deflection Chart for ODOT (31).

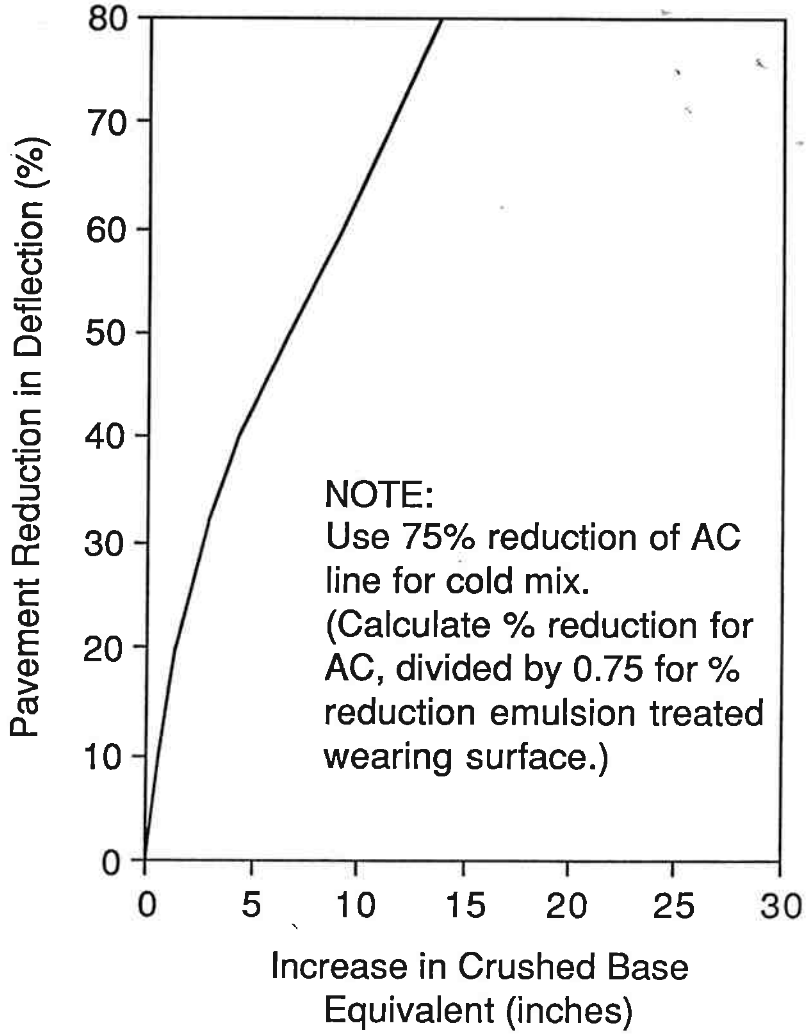


Figure 2.4. Reduction in Deflection from Pavement Overlays for ODOT (31).

Table 2.1. Advantages and Disadvantages of the Existing Overlay Method for Flexible Pavements.

	a) Advantages of Oregon DOT Method
1)	Method is easy to apply.
2)	Input data are easily obtainable.
	b) Disadvantages of Oregon DOT Method
1)	Does not consider remaining life of the pavement.
2)	Does not consider specific distresses (fatigue vs. rutting).

3) composite subgrade reaction (k),

4) load safety factor,

5) axle load distribution, and

6) expected number of axle-load repetitions during the design period.

Two different failure modes are considered: erosion and fatigue. Erosion related failure in the pavement occurs due to excessive pumping, erosion of foundation materials and joint faulting. This failure mode usually controls the design of medium-heavy volume roads without doweled joints and heavy-volume roads with doweled joints. Tandem-axle loads are more severe in the erosion analysis while single-axle loads are of more concern in the fatigue analysis. The fatigue analysis usually controls the design of low volume roads and dowel jointed medium volume roads. ODOT considers both the erosion analysis as well as the fatigue analysis in the design of overlays. Each analysis procedure is briefly discussed below.

Table 2.2 summarizes the procedure used by ODOT in the erosion analysis for the PCA method. Columns 1 and 3 are design input data. Column 2 is determined by multiplying the axle load by a load safety factor. In this analysis the erosion factor is first determined by estimating a k value from Table 2.3 for the existing pavement. Note the contribution of the surface layer is not considered in this analysis. A preliminary overlay slab thickness is then assumed and the erosion factor is determined from Table 2.4. Column 2 and the erosion factor are then used in Figure 2.5 to determine the allowable repetitions to failure for column 6 in Table 2.2. The percent damage for each axle load is calculated by dividing column 3 by column 6 and multiplying by 100. The damage values are summed at the bottom for both the

Table 2.2. Calculation of Pavement Thickness for PCA Procedure (38).

Project Design 1A, four-lane Interstate, rural

Trial thickness	9.5 in.	Doweled joints:	yes ✓ no ___
Subbase-subgrade k	130 pci	Concrete shoulder:	yes ___ no ✓
Modulus or rupture, M_R	650 psi	Design period	20 years
Load safety factor, LSF	1.2	4-in. untreated subbase	

Axle Load kips	Multiplied LSF (1.2)	Expected Repetitions	Fatigue Analysis: Allowable Repetitions	Fatigue Analysis: Fatigue Percent	Erosion Analysis: Allowable Repetitions	Erosion Analysis: Damage Percent
1	2	3	4	5	6	7
		8. Equivalent stress 206			10. Erosion factor 2.59	
		9. Stress ratio factor 0.317				
Single Axles						
30	36.0	6,310	27,000	23.3	1,500,000	0.4
28	33.6	14,690	77,000	19.1	2,200,000	0.7
26	31.2	30,140	230,000	13.1	3,500,000	0.9
24	26.8	64,410	1,200,000	5.4	5,900,000	1.1
22	26.4	106,900	Unlimited	0	11,000,000	1.0
20	24.0	235,800	"	0	23,000,000	1.0
18	21.6	307,200	"	0	64,000,000	0.5
16	19.2	472,500			Unlimited	0
14	16.8	586,900			"	0
12	14.4	1,837,000			"	0
		11. Equivalent stress 192			13. Erosion factor 2.79	
		12. Stress ratio factor 0.295				
Tandem Axles						
52	62.4	21,320	1,100,000	1.9	920,000	2.3
48	57.6	42,870	Unlimited	0	1,500,000	2.9
44	52.8	124,900	"	0	2,500,000	5.0
40	48.0	372,900	"	0	4,600,000	8.1
36	43.2	885,800			9,500,000	9.3
32	38.4	930,700			24,000,000	3.9
28	33.6	1,656,000			92,000,000	1.8
24	28.8	984,900			Unlimited	0
20	24.0	1,227,000			"	0
16	19.2	1,356,000				
			Total	62.8	Total	38.9

Table 2.3. Effect of Untreated and Treated Subbase on k Values (38).

a) k Values for Untreated Base

Subgrade k value, pci	Subbase k value, pci			
	4 in.	6 in.	9 in.	12 in.
50	65	75	85	110
100	130	140	160	190
200	220	230	270	320
300	320	330	370	430

b) k Values for Cement-Treated Subbases

Subgrade k value, pci	Subbase k value, pci			
	4 in.	6 in.	8 in.	10 in.
50	170	230	310	390
100	280	400	520	640
200	470	640	830	-

Table 2.4. Erosion Factors: Doweled Joints No Concrete Shoulder (38).

(Single axle/tandem axle)

Slab thickness, in.	k of subgrade-subbase, pci					
	50	100	200	300	500	700
4	3.74/3.83	3.73/3.79	3.72/3.75	3.71/3.73	3.70/3.70	3.68/3.67
4.5	3.59/3.70	3.57/3.65	3.56/3.61	3.55/3.58	3.54/3.55	3.52/3.53
5	3.45/3.58	3.43/3.52	3.42/3.48	3.41/3.45	3.40/3.42	3.38/3.40
5.5	3.33/3.47	3.31/3.41	3.29/3.36	3.28/3.33	3.27/3.30	3.26/3.28
6	3.22/3.38	3.19/3.31	3.18/3.26	3.17/3.23	3.15/3.20	3.14/3.17
6.5	3.11/3.29	3.09/3.22	3.07/3.16	3.06/3.13	3.05/3.10	3.03/3.07
7	3.02/3.21	2.99/3.14	2.97/3.08	2.96/3.05	2.95/3.01	2.94/2.98
7.5	2.93/3.14	2.91/3.06	2.88/3.00	2.87/2.97	2.86/2.93	2.84/2.90
8	2.85/3.07	2.82/2.99	2.80/2.93	2.79/2.89	2.77/2.85	2.76/2.82
8.5	2.77/3.01	2.74/2.93	2.72/2.86	2.71/2.82	2.69/2.78	2.68/2.75
9	2.70/2.96	2.67/2.87	2.65/2.80	2.63/2.76	2.62/2.71	2.61/2.68
9.5	2.63/2.90	2.60/2.81	2.58/2.74	2.56/2.70	2.55/2.65	2.54/2.62
10	2.56/2.85	2.54/2.76	2.51/2.68	2.50/2.64	2.48/2.59	2.47/2.56
10.5	2.50/2.81	2.47/2.71	2.45/2.63	2.44/2.59	2.42/2.54	2.41/2.51
11	2.44/2.76	2.42/2.67	2.39/2.58	2.38/2.54	2.36/2.49	2.36/2.45
11.5	2.38/2.72	2.36/2.62	2.33/2.54	2.32/2.49	2.30/2.44	2.29/2.40
12	2.33/2.68	2.30/2.58	2.28/2.49	2.26/2.44	2.25/2.39	2.23/2.36
12.5	2.28/2.64	2.25/2.54	2.23/2.45	2.21/2.40	2.19/2.35	2.18/2.31
13	2.23/2.61	2.20/2.50	2.18/2.41	2.16/2.36	2.14/2.30	2.13/2.27
13.5	2.18/2.57	2.15/2.47	2.13/2.37	2.11/2.32	2.09/2.26	2.08/2.23
14	2.13/2.54	2.11/2.43	2.08/2.34	2.07/2.29	2.05.2.23	2.03/2.19

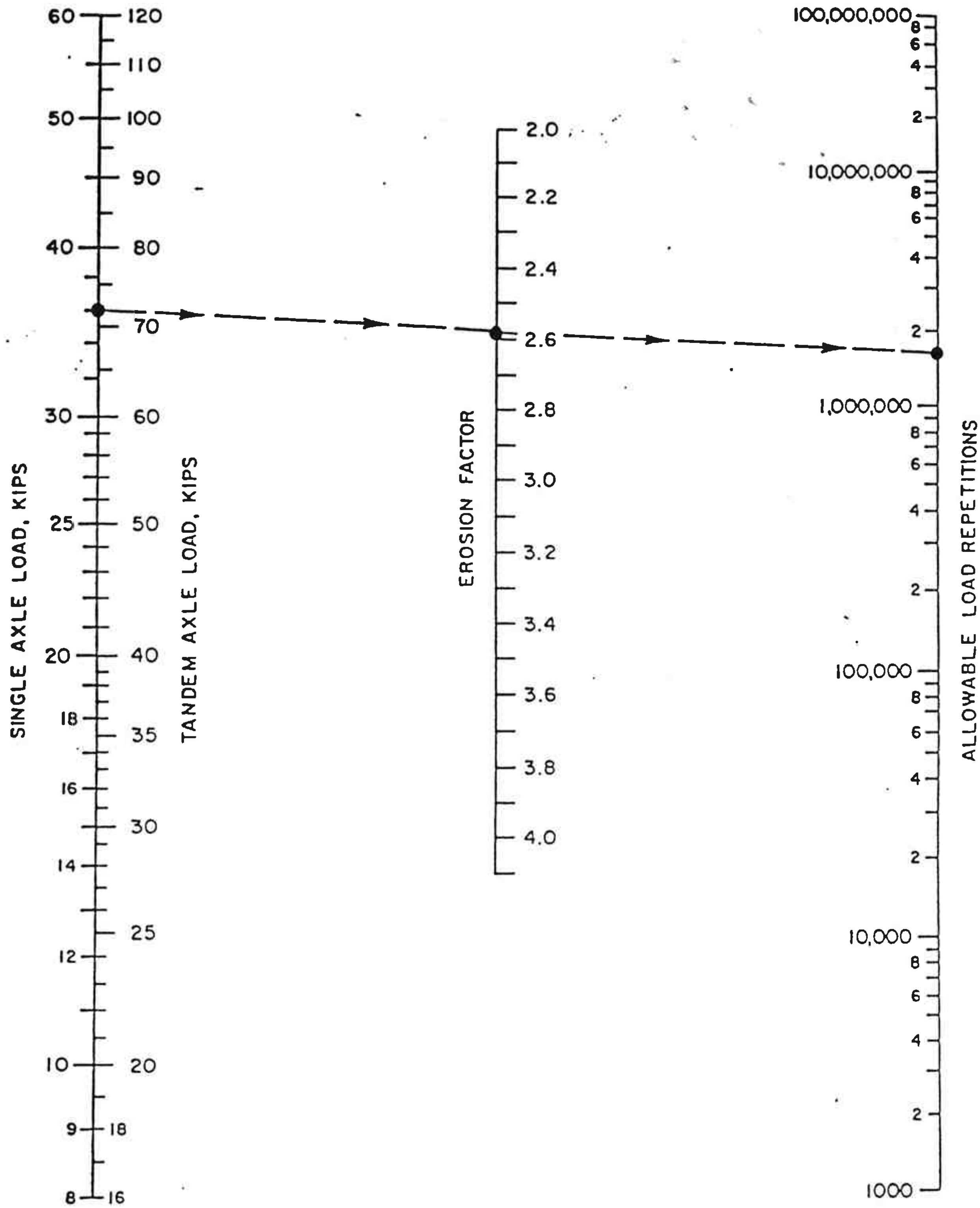


Figure 2.5. Erosion Analysis - Allowable Load Repetitions Based on Erosion Factor (No Concrete Shoulder) (38).

single and tandem axle load data. If the sum is greater than 100 then a greater slab thickness must be assumed for determining the erosion factor. ODOT usually increases the thicknesses in increments of 1 in. (2.54 cm), so the first value with a percent damage of less than 100 is used.

Table 2.2 is used for the fatigue analysis as well. In this analysis columns 4 and 5 are completed to check the summed value against fatigue damage. The allowable repetitions (column 4) are determined from column 3 and the stress ratio by the use of Figure 2.6. The stress ratio in Figure 2.6 is computed by dividing the equivalent edge stress by the modulus of rupture for the concrete. The equivalent stress is determined from Table 2.5. The consumed fatigue life is calculated by dividing expected repetitions by the allowable repetitions and multiplying by 100. As in the previous procedure this value is summed at the bottom for the single and tandem axle loads and must fall less than the value of 100.

The AASHTO Method (1). The slab thickness is calculated using the nomograph given in Figure 2.7. The following assumptions form the basis for the nomograph:

1) The equations developed from the AASHTO Road Test represent the relationships that exist between loss in serviceability, traffic, pavement materials, and thickness.
2) The equations are valid for any roadbed soil by adjusting the subgrade reaction, k.
3) All traffic may be converted to equivalent 18k Equivalent Single Axle Load (ESAL).
4) The equations developed for traffic during the two-year test period may be applied over an extended period of time.

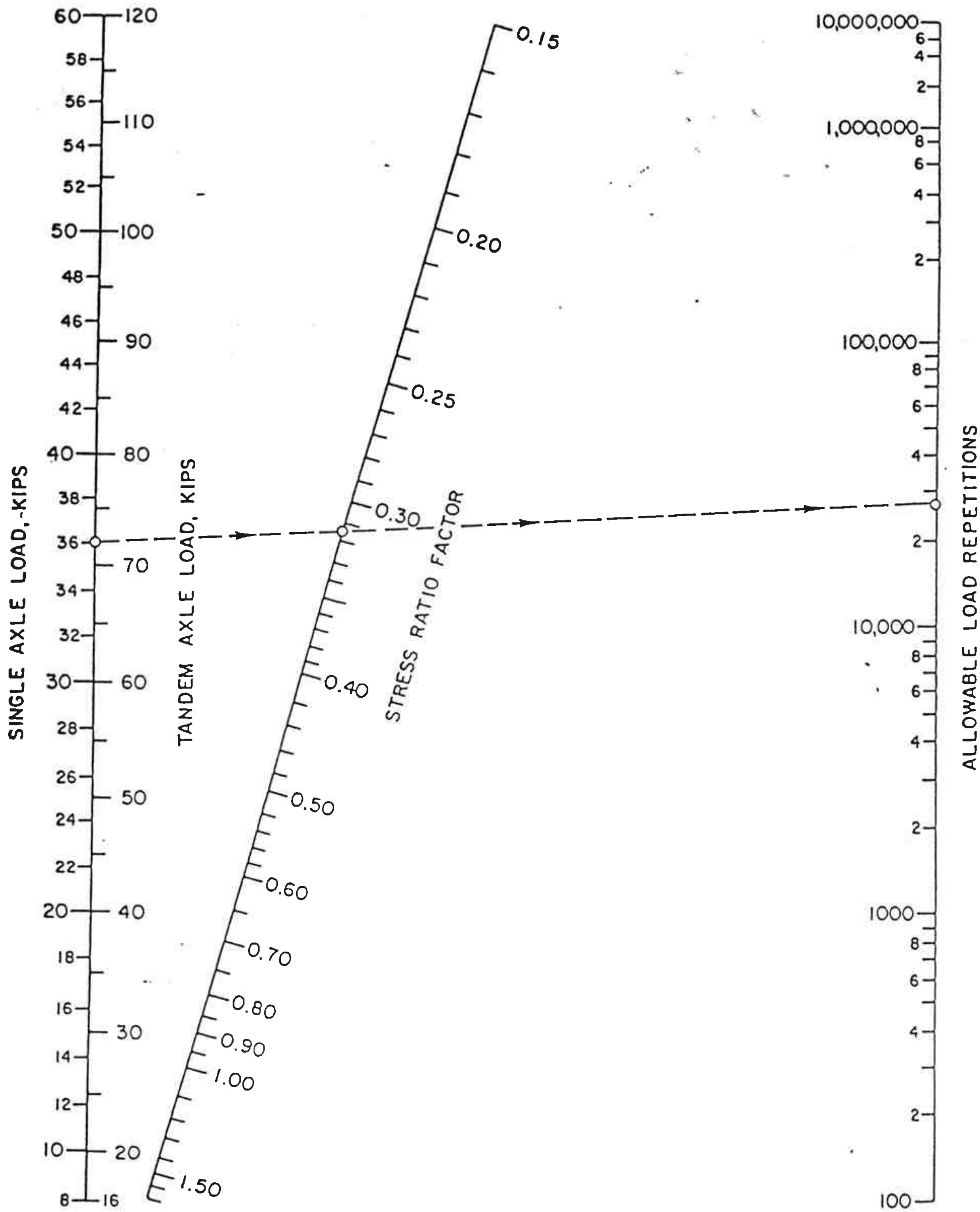


Figure 2.6. Fatigue Analysis - Allowable Load Repetitions Based on Stress Ratio Factor (with/without Concrete Shoulder) (38).

Table 2.5. Equivalent Stress - No Concrete Shoulder (38).

(Single axle/tandem axle)

Slab thickness, in.	k of subgrade-subbase, pci						
	50	100	150	200	300	500	700
4	825/679	726/585	671/542	634/516	584/486	523/457	484/443
4.5	699/586	616/500	571/460	540/435	498/406	448/378	417/363
5	602/516	531/436	493/399	467/376	432/349	390/321	363/307
5.5	526/461	464/387	431/353	409/331	379/305	343/278	320/264
6	465/416	411/348	382/316	362/296	336/271	304/246	285/232
6.5	417/380	367/317	341/286	324/267	300/244	273/220	256/207
7	375/349	331/290	307/262	292/244	271/222	246/199	231/186
7.5	340/323	300/268	279/241	265/224	246/203	224/181	210/169
8	311/300	274/249	255/223	242/208	225/188	205/167	192/155
8.5	285/281	252/232	234/208	222/193	206/174	188/154	177/143
9	264/264	232/218	216/195	205/181	190/163	174/144	163/133
9.5	245/248	215/205	200/183	190/170	176/153	161/134	151/124
10	228/235	200/193	186/173	177/160	164/144	150/126	141/117
10.5	213/222	187/183	174/164	165/151	153/136	140/119	132/110
11	200/211	175/174	163/155	154/143	144/129	131/113	123/104
11.5	188/201	165/165	153/148	145/136	135/122	123/107	116/98
12	177/192	155/158	144/141	137/130	127/116	116/102	109/93
12.5	168/183	147/151	136/135	129/124	120/111	109/97	103/89
13	159/176	139/144	129/129	122/119	113/106	103/93	97/85
13.5	152/168	132/138	122/123	116/114	107/102	98/89	92/81
14	144/162	125/133	116/118	110/109	102/98	93/85	88/78

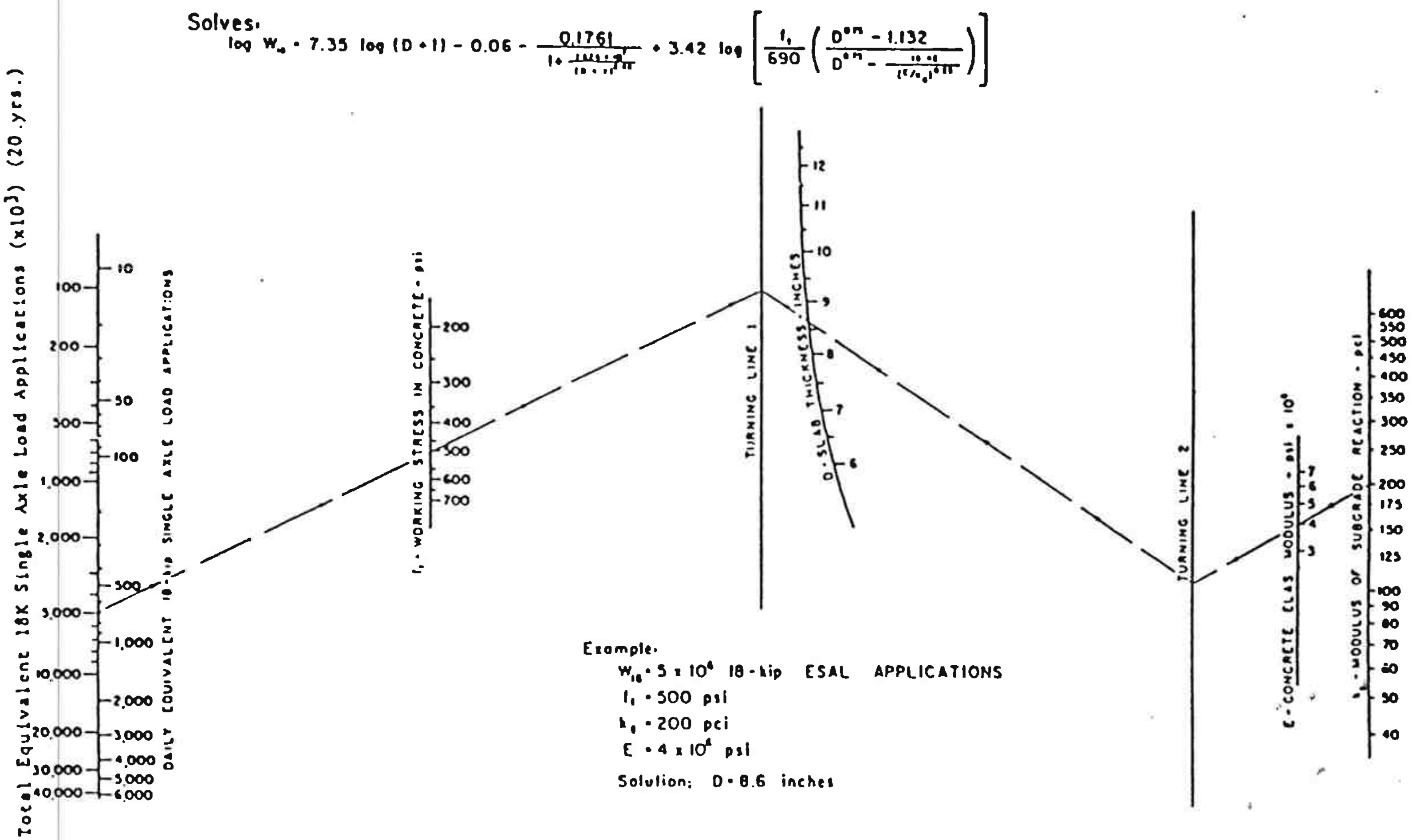


Figure 2.7. AASHTO Design Chart for Rigid Pavements (1).

5) The equations developed for the portland cement concrete used at the AASHTO Road Test may be extended to other types of portland cement concrete by changing the modulus of elasticity of the concrete.

As indicated in Figure 2.7, the thickness of the pavement slab is a function of the estimated traffic (18k ESAL), working stress (psi), the subgrade modulus (k), and the modulus of elasticity of the concrete (psi). The working stress is calculated by the following equation:

$$f_t = S_c/C \tag{2.1}$$

where:

S_c = 28-day flexural strength using third point loading, and

C = constant to determine working stress and represents the level of confidence in the design. A value of 1.33 is usually recommended.

The subgrade k value is estimated based on previous experience or by correlation with other tests. The value k takes into the account the strength of the entire pavement structure, and not just the subgrade. Therefore, an assessment must be made of the strength of both the underlying materials and the asphalt layer.

The modulus of elasticity may be computed from a static compression test on a cylinder, or it must be estimated. The slab thickness determined from Figure 2.7 is normally rounded off to the nearest inch. This value is the design overlay thickness.

The final design is determined using engineering judgment by comparing the results of both PCA and AASHTO methods. Advantages and disadvantages of these methods are shown in Table 2.6.

2.1.3 Other Combinations

Very few overlays are placed over rigid pavements. In most instances when a rigid pavement becomes fatigued, reconstruction is usually selected. Asphalt overlays over rigid pavements are occasionally performed by assigning crushed base equivalents to the existing pavement layers, and then designing it as a new flexible pavement. However, in most instances the overlay thickness is controlled by reflection cracking criteria. This procedure does not use the deflection equipment to assess the pavement structure.

Currently, life cycle cost evaluations are performed in Oregon to determine the most economical rehabilitation recommendations (29).

2.2 Proposed AASHTO Method (AASHTO, 1986) (2)

This procedure is based on the serviceability-traffic (performance) and structural capacity-traffic relationships developed at the AASHTO Road Test. Determination of an overlay is accomplished by using a deficiency approach. Various strategies can be evaluated; the final selection is based on a life cycle cost analysis plus other considerations. The remaining life for the existing pavement for the desired level of serviceability can also be determined from nondestructive tests (NDT).

The basic design concept is illustrated in Figure 2.8, where the following definitions apply:

1) P_o = the initial serviceability of the original pavement;

Table 2.6. Advantages and Disadvantages of Existing Concrete Overlay Method for Rigid Pavements.

a) Advantages

1) Methods are easy to apply.

2) Parameters (except for k) are easy to obtain.

b) Disadvantages

1) The two methods do not give consistent results. For Interstates, AASHTO provides for a thicker overlay and the PCA yields a thicker overlay for lower volume roads.

2) Neither method considers the remaining life.

3) It is difficult to obtain accurate values of k.

4) Deflection values are not incorporated into the design.

5) AASHTO scale for traffic is not adequate for present traffic levels (Interstate projects have traffic volumes as high as 60×10^6 ESAL's).

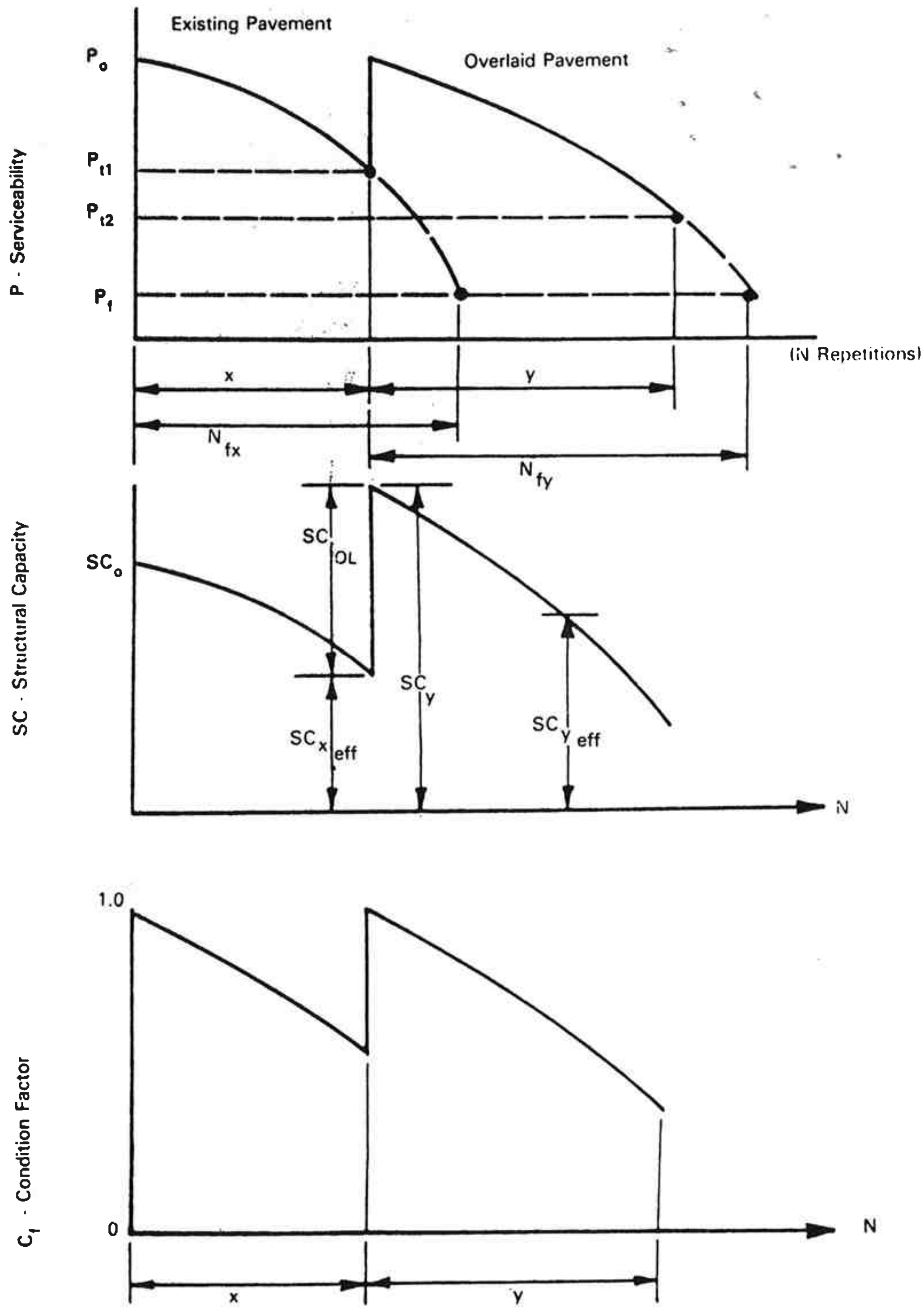


Figure 2.8. Relationship Between Serviceability - Capacity Condition Factor and Traffic (2).

2) P_{t1} = terminal serviceability of the existing pavement prior to the overlay;

3) P_{t2} = the terminal serviceability desired from the overlaid pavement once the traffic loads have been applied; and

4) P_f = the ultimate failure serviceability corresponding to a completely damaged pavement.

In the proposed method the P_f value has been set as 2.0 while P_{t1} and P_{t2} are input considerations of the designer. The values of N_{FX} and N_{FY} represent the total number of repetitions necessary for the original pavement and the overlaid pavement to reach failure (P_f = 2.0) respectively.

The structural capacity necessary to support the overlay traffic (SC_y) is illustrated in Figure 2.8. The initial structural capacity is noted by SC_o which gradually reduces to an "effective capacity" (SC_{xeff}) prior to the overlay. These two terms are related by a condition factor, "C_x" as follows:

$$SC_{xeff} = C_x SC_o \tag{2.2}$$

The basis of the method with remaining life concepts included takes the form:

$$SC_{OL}^n = SC_y^n - F_{RL}(SC_{xeff})^n \tag{2.3}$$

where:

SC_{OL} = Structural capacity, required in overlay thickness for overlay traffic,

SC_y = Structural capacity necessary for overlay traffic,

SC_{xeff} = Effective structural capacity of existing pavement structure,

F_{RL} = The remaining life factor accounting for damage in the existing pavement and the desired degree of damage at the end of the overlay traffic (≤ 1.0), and

n = Constant dependent upon the type of pavement system used. Asphalt and portland cement (bonded) = 1, portland cement concrete (unbonded) = 2.

The structural capacity (SC) represents a strength parameter for all types of pavement. In the following discussion for flexible pavements the structural number, SN, and for rigid pavements the required PCC slab thickness, D, is used in place of SC.

2.2.1 Steps for Overlay Design

Seven steps are necessary before an overlay design may be developed. Detailed design steps and relationships are contained in Appendix C. The seven basic steps are briefly discussed in the following paragraphs:

STEP 1: Analysis Unit Delineation

The design philosophy considers two alternatives regarding availability and accuracy of historic information: accurate historic data are either available or unavailable.

If a project unit lacks the available data, NDT equipment is employed to obtain deflections every 300 to 500 feet (91.4 to 152.4 m) in the outer wheel path of the lane adjacent to the shoulder. This information is utilized to verify, modify, or supply data for the various units. If a project length has sufficient data, then deflection tests can be performed randomly (10 to 15 tests/unit).

STEP 2: Traffic Analysis

Traffic analysis has two important components with "x" being the cumulative 18k ESAL repetitions until an overlay is placed and "y" being the cumulative 18k ESAL repetitions expected in the future for the overlay. The structural overlay can not be designed until the engineer determines the future traffic repetitions that are anticipated over the life of the overlay. In some instances, historical data on the traffic loads the existing pavement has already endured is required. This information is needed to aid in the determination of the remaining life factor (F_{RL}).

STEP 3: Materials and Environmental Study

The objective of this step is to determine the best available estimates of the properties of (a) the existing pavement layers, including the subgrade; and (b) the overlay layer. The elastic modulus of each layer is the most important property and can be determined by destructive testing or by backcalculation. The latter technique uses results of NDT deflection basin measurements together with a multilayer elastic computer program to solve for the layer moduli (and subsequently the structural layer coefficients a_i). The deflection basin (Figure 2.9) is defined by the deflection values measured at various radial offset values (r_n). The basis of this method is derived from the assumption that there exists a unique set of layer moduli ($E_1, E_2, \ldots E_n$) as a result of which the predicted deflections (basin) will yield the exact measured deflection basin.

If backcalculation techniques are not used, then the subgrade modulus can be directly estimated from the AASHTO equation:

$$E_{SG} = (PS_f)/(d_r r) \tag{2.4}$$

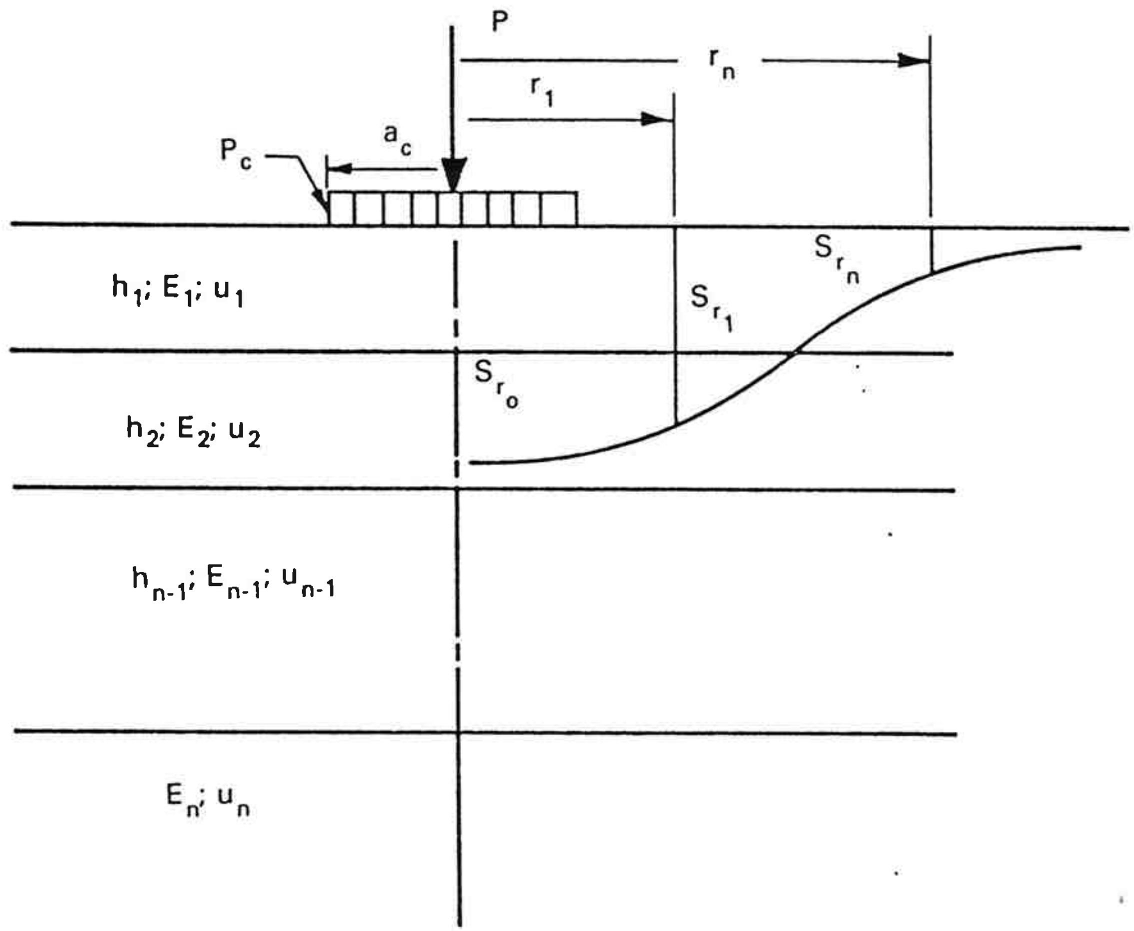


Figure 2.9. Deflection Basin Analysis for Estimating In Situ Moduli (2).

where:

E_{SG} = in situ modulus of elasticity (psi) of subgrade layer,

P = dynamic load (lb) 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

S_f = subgrade modulus prediction factor which is a function of Poisson's ratio.

These values are valid for $r/a_e > 1.0$, where the distance a_e is determined by:

$$a_e = a_c/F_b \tag{2.5}$$

where:

a_c = radius (inches) of NDT load plate,

F_b = deflection factor which is a function of Poisson's ratio of the subgrade and H_e/a_c, and

H_e/a_c = the ratio of the pavement's effective thickness to the plate radius.

Each pavement system has its own unique minimum value of a_e. As the pavement layer becomes thicker and/or stiffer a_e increases. If the ratio $r/a_e \leq 1.0$ then the S_f factor is a function of r/a_e rather than a constant.

STEP 4: Effective Structural Capacity Analysis (SC_{xeff})

The primary method for estimating the structural capacity is through the use of the deflection basin obtained from the NDT equipment. NDT Method 1 utilizes the deflection basin data to calculate layer moduli as opposed to NDT Method 2 which uses the maximum deflection value.

For rigid pavements, the resulting effective structural capacity, SC_{xeff}, is equal to the effective PCC thickness, D_{xeff}. The PCC effective layer moduli (E_{PCC}) reflects the loss in slab support due to cracking and partial support loss. Knowing E_{PCC} (from destructive testing or from the NDT analysis) and the existing pavement thickness (D_o), D_{xeff} can be determined. If NDT equipment is not available, several other techniques are available including: (a) Visual Condition Factor Approach, (b) Nominal Size of PCC Slab Fragments, and (c) Remaining Life Approach.

When flexible pavement systems are evaluated, the effective structural number (SN_{xeff}) represents the effective structural capacity. This can be determined in one of two ways. NDT Method 1 estimates the effective layer moduli for each of the pavement layers (asphalt concrete, base, etc.) using deflection basin measurements. The structural layer coefficients are determined from the modulus values. For asphalt materials, it is important to adjust the deflections to a standard temperature of 70°F.

With NDT Method 2, the maximum measured pavement deflection (d_o) and the characteristics of the particular NDT equipment are used to estimate the in situ subgrade modulus (E_{SG}). The final design subgrade modulus must be adjusted to account for environmental influences. The maximum deflection value used is also adjusted to the standard temperature. Using an iterative procedure, SN_{xeff} is estimated from the maximum deflection, thickness of pavement structure, and E_{SG}.

STEP 5: Future Structural Capacity Analysis (SC_y)

The analysis assumes that the structural capacity of a new pavement is determined over the same existing subgrade for the required future repetitions. In essence, a new pavement is being designed. The subgrade support value should be the design value obtained in Step 3 for either pavement type. The appropriate P_{t2} value (terminal serviceability value for overlay) is selected by the designer and the design traffic repetitions developed in Step 2 are used in this analysis. The structural capacity is then determined from Figure 2.10. The structural capacity for rigid overlay is determined in the same manner (Appendix C).

STEP 6: Remaining Life Factor Determination (F_{RL})

The next step is the determination of the remaining life factor (F_{RL}) which is multiplied by the effective capacity parameter (SN_{xeff} or D_{xeff}) to obtain a more realistic assessment of the effective capacity during the overlay period. Both the remaining life of the existing pavement prior to the overlay, R_{LX}, and the remaining life of the overlaid pavement system after overlay traffic has been reached, R_{LY}, must be known to determine the remaining life of the combined structure. The difficulty arises in estimating the remaining life of the existing pavement prior to the overlay due to uncertainties in historical traffic and remaining composite strength. Five possible methods are outlined to assist in this task:

1) NDT Approach: Results from the deflection testing can be used to determine the R_{LX} value. Knowing the initial structural capacity value, the pavement condition factor can be computed from:

$$C_x = SN_{xeff}/SN_o \quad \text{or} \quad C_y = D_{xeff}/D_o \tag{2.6}$$

Once the C_x value is calculated, the remaining life can be estimated.

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_o + 9.36*\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32*\log_{10} M_R - 8.07$$

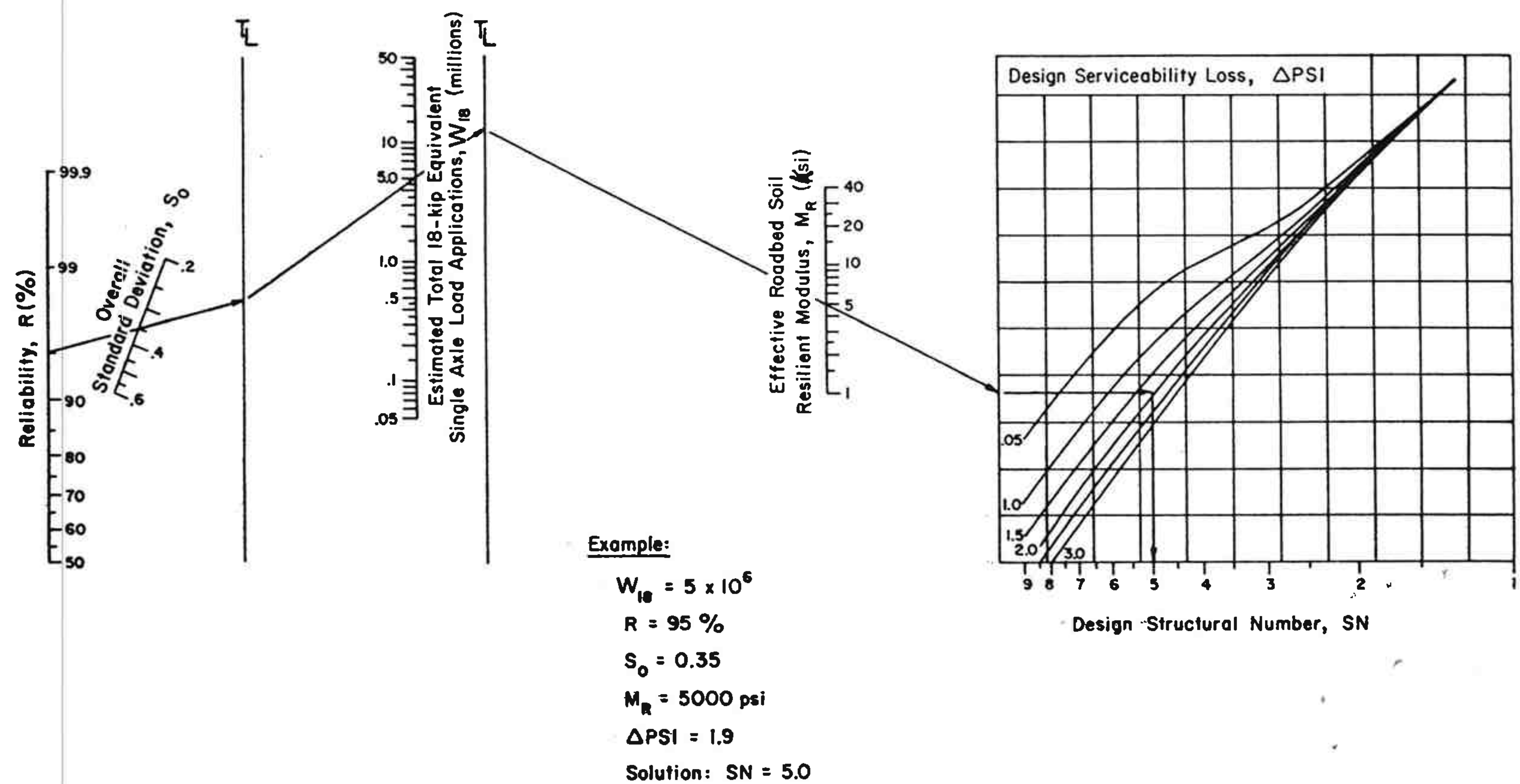


Figure 2.10. Design Chart for Flexible Pavements Based on Using Mean Values for Each Input (2).

2) Traffic Approach: Accurate information for the previous traffic history of the existing pavement must be available to use this approach. The equivalent number of repetitions (N_{FX}) to reach failure serviceability (P_f = 2.0) can be estimated. The R_{LX} value can then be computed using

$$R_{LX} = (N_{FX} - x)/N_{FX} \tag{2.7}$$

3) Time Approach: The R_{LX} value can be approximated from the following factors if specific traffic information is not readily available:

t = time (yr) highway section has been in service,

T_f = best estimate of probable time (yr) a particular pavement type typically lasts before an overlay is needed, and

r_g = annual traffic growth rate of the highway section.

4) Serviceability Approach: The R_{LX} of an existing pavement may be estimated if the present serviceability index and initial structural capacity is known.

5) Visual Condition Survey Approach: A condition factor C_x can be formulated from condition surveys. Individual pavement condition factors (C_v) are obtained from Table 2.7. Using a weighted procedure based upon the layer thickness, the pavement condition factor, C_x, may be calculated.

$$C_x = h_1C_{v1} + h_2C_{v2} + \ldots h_nC_{vn}/h_1 + h_2 + \ldots h_n \tag{2.8}$$

Table 2.7. Summary of Visual (C_V) and Structural (C_X) Condition Values (2).

Layer Type	Pavement Condition	C_V Visual Condition Factor Range	C_X Structural Condition Factor Value
Asphaltic	1. Asphalt layers that are sound, stable, uncracked and have little to no deformation in the wheel paths.	0.9-1.0	.95
	2. Asphalt layers that exhibit some intermittent cracking with slight to moderate wheel path deformation but are still stable.	0.7-0.9	.85
	3. Asphalt layers that exhibit some moderate to high cracking, have raveling or aggregate degradation and show moderate to high deformation in wheel path.	0.5-0.7	.70
	4. Asphalt layers that show very heavy (extensive) cracking, considerable raveling or degradation and very appreciable wheel path deformations.	0.3-0.5	.60
PCC	1. PCC pavement that is uncracked, stable and undersealed, exhibiting no evidence of pumping.	0.9-1.0	.95
	2. PCC pavement that is stable and undersealed but shows some initial cracking (with tight, nonworking cracks) and no evidence of pumping.	0.7-0.9	.85
	3. PCC pavement that is appreciably cracked or faulted with signs of progressive crack deterioration: slab fragments may range in size from 1 to 4 sq. yds., pumping may be present.	0.5-0.7	.70
	4. PCC pavement that is very badly cracked or shattered into fragments 2 to 3 ft in maximum size.	0.3-0.5	.60
Pozzolanic Base/ Subbase	1. Chemically stabilized bases (CTB, LCF, ...) that are relatively crack free, stable and show no evidence of pumping.	0.9-1.0	.95
	2. Chemically stabilized bases (CTB, LCF, ...) that have developed very strong pattern or fatigue cracking, with wide and working cracks that are progressive in nature; evidence of pumping or other causes of instability may be present.	0.3-0.5	.60
Granular Base/ Subbase	1. Unbound granular layers showing no evidence of shear or densification distress, reasonably identical physical properties as when constructed and existing at the same "normal" moisture density conditions as when constructed.	0.9-1.0	.95
	2. Visible evidence of significant distress within layers (shear or densification), aggregate properties have changed significantly due to abrasion, intrusion of fines from subgrade or pumping, and/or significant change in in-situ moisture caused by surface infiltration or other sources.	0.3-0.5	.60

Special Notes:
1. The visual condition factor, C_V, is related to the structural condition factor, C_X, by: $C_V = C_X^2$.
2. The structural condition factor, C_X, and <u>not</u> the C_V value, is the variable used in the structural overlay design equation (for all overlay-existing pavement types). It is defined by $SC_{xeff} = C_x SC_o$.

Although all the above procedures are valid, the procedure utilizing the NDT deflections should provide a better quantitative assessment of the existing structural capacity. Advantages and disadvantages of each of the remaining life approaches are given in Table 2.8.

The remaining life of the overlaid pavement (R_{LY}) is computed from the design input parameters selected by the engineer as follows:

$$R_{LY} = (N_{FY} - y)/N_{FY} \tag{2.9}$$

From Step 5 the ultimate number of repetitions to failure N_{FY} (from Figure 2.10) that the pavement SC_y would be subjected to is derived from the knowledge of the design overlay traffic (y) and the SC_y (structural capacity) to yield P_{t2} after "y" repetitions. Knowing both R_{LX} and R_{LY} the remaining life factor F_{RL} is determined.

STEP 7: Determine Overlay Thickness

The final step is to calculate the SN_{OL} or D_{OL} from one of the equations in Table 2.9 once all the other factors are calculated.

2.3 Evaluation of the AASHTO Method

The AASHTO method can be best evaluated by looking at its components separately. Each step outlines the inputs necessary to implement the AASHTO procedure.

The first step involves identifying the homogeneous sections of the highway to be tested for deflection. This is a function of the distress of certain areas and the amount of available historic data for that particular highway. This step is routinely performed for every overlay procedure so there should be no difficulty in assigning these sections.

Table 2.8. Advantages/Disadvantages of Remaining Life Approaches (2).

Remaining Life Approach	Zone 1 (New to Slightly Damaged)	Zone II (Fair to Significantly Damaged)
NDT	May be difficult to accurately estimate R_{Lx} by C_f factor due to impact of deflection variability. Estimates of structural capacity may be more indicative of actual initial in-situ value (SC_o).	May be most accurate method for estimating R_{Lx} value from effective capacity measurements. Deflection response is a measure of of actual in situ properties of all material layers and as a result should be the best estimate of the effective capacity. Measured deflections should accurately reflect reduced load spreading capacity of pavement system due to structural cracking and/or increased moisture infiltration in unbound layers.
Traffic	May be one of the best approaches to utilize especially if pavement is relatively new (undamaged). This is because traffic repetition projections should be rather easy to estimate and be relatively accurate. Accuracy of the approach may be enhanced by combining with NDT to estimate the initial in-situ pavement capacity and support values to determine accurate prediction of the N_{fx} value used in the R_{Lx} study.	Accuracy of the approach is highly dependent upon the ability to accurately predict previous traffic history <u>and</u> the ultimate repetitions to failure ($p = 2.0$). With increasing time and traffic repetitions, the ability to accurately predict both of these parameters logically decreases unless accurate historic data exist.
Time	Like the Traffic Approach remarks, this approach may provide a realistic estimate of the remaining life provided accurate estimates (historic data) are available for typical failure lives and traffic growth rates. In general, the accuracy of this predictive approach should be less than than for the Traffic Approach.	Comments noted in Zone I are also applicable for this approach used in Zone II.
Serviceability	May be difficult to accurately estimate R_{Lx} due to the large sensitivity of R_{Lx} to small changes in serviceability at high p values. There may be significant influence/difficulty in delineating serviceability loss components due to environmental (nonload) and structural sources.	While the sensitivity of serviceability to R_{Lx} estimates is better in this Zone compared to Zone I, inaccuracies between the predictive model for p-traffic and the actual field performance may lead to extremely inaccurate estimates of R_{Lx}. Comments regarding the impact of environmentally induced serviceability losses noted in Zone I are also applicable here. With time (damage), the ability to determine the precise in situ initial structural capacity (SC_o) is decreased unless accurate historic data are available.
Visual Condition Survey	Most comments noted for the NDT and Serviceability Approaches are also applicable to this method. The anticipated accuracy of the visual approach is decreased over other procedures due to the subjectiveness (variability) used to estimate condition factors.	Approach obviously suffers from subjectiveness associated with selecting an appropriate C_f value. It should be recognized that it is nearly impossible to precisely estimate condition factors of pavement layers below the surface unless destructive test pits are made to examine underlying pavement layers and in-situ testing gives an indication of significantly increased moisture (strength reductions).

Table 2.9. Specific Overlay Equation Form Utilized (2).

Type Overlay	Type Existing Pavement	Specific Equation	Conditions/Remarks
Flexible	Flexible	$SN_{oL} = SN_y - F_{RL}\ SN_{xeff}$	SC = SN; n = 1.0
Flexible	Rigid	$SN_{oL} = SN_y - F_{RL}\ SN_{xeff}$	SC = SN; n = 1.0 (see Appendix C for equations used
Rigid	Flexible	$D_{oL} = D_y$ (see remarks)	Treat overlay analysis as new rigid pavement design using existing flexible pavement as new foundation (subgrade)
Rigid	Rigid	$D_{oL} = D_y - F_{RL}\ (D_{xeff})$	SC = D; n = 1.0 (Bonded Overlay)
		$D_{oL}^{1.4} = D_y^{1.4} - F_{RL}\ (D_{xeff})^{1.4}$	SC = D; n = 1.4 (Partial Bond Overlay)
		$D_{oL}^{2} = D_y^{2} - F_{RL}\ (D_{xeff})^{2}$	SC = D; n = 2.0 (Unbonded Overlay)

General Structural Capacity Form: $SC_{oL}^{n} = SC_y^{n} - F_{RL}\ (SC_{xeff})^{n}$

The second step involves the determination of traffic estimates 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. If needed, however, estimates of previous traffic may be difficult to obtain, especially for older low volume roads.

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 are calculated knowing the variables of the NDT equipment and the associated deflection values for the applied load. The moduli values can be backcalculated using computer programs such as BISDEF, ELSDEF or MODCOMP2. 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 some limitations for programs which may affect the degree of reliability obtained from the calculated layer moduli values. However, some programs e.g. BISDEF, provide much closer estimations of the pavement layer moduli if the range for the material is well bracketed. This may involve taking cores and performing laboratory tests to obtain an idea of the approximate moduli value for each layer.

The effective structural capacity of the existing pavement is determined in Step 4. This is dependent upon the type of structure to be overlaid. For an existing PCC pavement, 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 coring. For flexible pavements, SN_{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 coring. The sum of the product of layer coefficients and the thickness for each layer will yield the structural number of the pavement.

Step 5 determines the future structural capacity of the pavement and is the equivalent of a new structural design. This value is determined either by equations or through the use of nomographs. The equations include a determination of traffic from Step 2, reliability and standard deviation chosen by the designer, the subgrade modulus obtained in Step 3, and desired serviceability levels. The reliability factor and standard deviation (level of confidence that a pavement will not fail within in a specified time) are determined from the 1986 AASHTO guide (2). Once a value is selected, it will probably be a constant for all subsequent projects. The last input value needed is the change in the present serviceability index value. There should be little or no difficulty encountered in this step.

The next step in the overlay design process is the calculation of the remaining life factor. Several methods are presented for the determination of the remaining life of the in situ pavement and the future overlaid pavement. AASHTO recommends use of the NDT approach for fatigued pavements and the traffic approach for newer pavements. It should be noted, there may be a significant difference in F_{RL} values depending upon which method is chosen for determination.

The last step is substituting the calculated values into the appropriate overlay design equation. Once the SN_{OL} is determined it is divided by the layer coefficient (a = .44 for asphalt overlay) to find the required overlay thickness. D_{OL} is the required slab thickness for portland cement concrete overlays.

2.4 Summary

The 1986 AASHTO procedure has many possible benefits (Table 2.10). Therefore, it was studied as the proposed overlay procedure for Oregon. The AASHTO method should provide for a more reliable and efficient overlay strategy if these new input values can be estimated with a high degree of confidence, and accurately implemented into the design equations.

Table 2.10. Reported Benefits of AASHTO.

1) It provides an overlay design procedure for all pavement types.
2) NDT equipment is used to estimate existing pavement layer properties.
3) It accounts for remaining life in the design.
4) It incorporates reliability (the probability the design life will be achieved).
5) It includes life cycle cost analysis.

3.0 DEVELOPMENT OF IMPROVED FRAMEWORK

This chapter identifies the requirements for a new overlay design procedure and the required input data. Some of the input data may be more easily obtained than others. For this reason, a critical review of the proposed AASHTO method was undertaken.

This chapter addresses the potential problems of the AASHTO method which may be encountered. Input values and assumptions may be simplified to provide for a method which will facilitate the use of the proposed overlay design procedure. Finally, flowcharts are provided for an overview of the procedure.

3.1 Requirements of Improved Method

The requirements for the improved method were discussed with ODOT and are summarized in Table 3.1. Important considerations for each requirement are discussed below. Recommendations and concerns are also addressed.

3.1.1 Simplicity

The new method must be easily implemented. If the data are too difficult or too expensive to obtain the procedure will not be fully utilized. If the equations or theory are too complicated or time consuming, accurate results may be compromised. The users should fully understand the mechanics of the procedure, retain full documentation for future modifications, and train new personnel to ensure accurate results and appropriate utilization over time. In the past, procedures have been followed without a complete understanding or documentation of design concepts and relationships. The new procedure should assist the designer in providing for a more effective overlay design without causing significant difficulties in the manipulation of the equations. For this reason computerization is recommended. The program should be user-

Table 3.1. Requirements for an Improved Overlay Method.

Item	General Considerations
Simplicity	1) ease of use 2) computerization
Evaluation of Layer's Contribution	1) direct tests 2) backcalculation
Develop k Value for PCC Overlays	1) existing flexible 2) existing rigid
Better Method for Overlays Over Existing Concrete Pavements	1) bonding 2) break 'n' seat 3) reliability
Remaining Life	1) calculation of R_{LX} 2) life cycle cost 3) traffic

friendly to minimize complications. Full documentation will be provided so adjustments can be made to the program as necessary. The designer should realize the program is no substitute for sound engineering judgment. A thorough understanding of the mechanics of the program and the procedure is necessary to eliminate inaccurate conclusions resulting from faulty input.

3.1.2 Evaluation of Layer's Contribution

Presently, the in situ pavement is evaluated as a whole from the maximum surface deflection value. The current method does not evaluate the individual layer and its contribution to the pavement structure. However, this information could be valuable. If poor drainage or frost action has damaged underlying layers, overlays will not be as effective in solving the problem. The AASHTO procedure suggests using back calculation techniques to determine the modulus of each layer from specifically located sensors.

3.1.3 Develop k Value for PCC Overlays

The modulus of subgrade reaction k is needed for all portland cement concrete pavement designs. Currently, ODOT estimates the k value of the in situ pavement from thicknesses of the pavement structure. These values may not give enough credit to structurally sound pavements. Better methods of estimating k need to be developed. AASHTO uses NDT equipment to obtain deflections of the pavement structure. These deflections are normalized and then correlated with a strength parameter to determine an appropriate k for flexible pavements.

The k value used for rigid existing pavements should be the same as for the original PCC design according to AASHTO. In this case AASHTO gives credit to the existing slab thickness.

3.1.4 Better Method for Overlays Over Existing Portland Cement Concrete

The Oregon Department of Transportation does not have an overlay design procedure which uses deflection data to evaluate rigid pavements. Instead, either a component analysis is performed or reflection cracking criteria are used. For the component analysis, each pavement layer is assigned a crushed base equivalency, and these are summed for the entire structure. A flexible design then includes this total crushed base and it is designed as a new pavement. For reflection cracking, a minimum thickness approach is used.

Two alternatives are available for flexible overlays over existing portland cement concrete pavements: break and seat or a normal overlay. A normal structural overlay does not consider the possibility of reflective cracking in the design procedure. Therefore, minimum overlay thicknesses for reducing reflective cracking must be obtained from tables. These minimum thicknesses are a function of slab length and temperature differential for a region.

The break and seat approach (not yet used in Oregon) has the potential to reduce reflective cracking. The existing slabs are fractured into 24 to 42 in. (61 to 122 cm) pieces. A heavy roller compacts these large pieces to ensure that a firm seat is obtained before the asphalt overlay is placed. Reflective cracking is minimized since the pavement structure is basically destroyed. The effective slab length is reduced so the pavement behaves like a semi-rigid to flexible pavement. Caltrans applies a geotextile fabric between PCC pavement and the asphalt overlay (40). There are two methods for determining SN_{xeff}: (a) estimating nominal crack spacing, and (b) estimating post cracking. The nominal crack spacing assumes a fragment size of 30 in. (76.2 cm). From this, the SN_{xeff} can be determined by:

$$SN_{xeff} = 0.40D_o + SN_{xeff\text{-}rp} \quad (3.1)$$

where:

D_o = original thickness (in.) of PCC, and

$SN_{xeff\text{-}rp}$ = $\Sigma\, a_i h_i$ (excluding the PCC layer).

The post cracking approach employs the use of the NDT equipment to determine the effective modulus of the broken PCC layer. The structural layer coefficient can then be determined and multiplied by D_o and added to $SN_{xeff\text{-}rp}$ to obtain SN_{xeff} (2).

Portland cement concrete overlays over existing portland cement concrete pavements involve various bonding options. The most important considerations in achieving a good bond are

1) strength and integrity of the existing pavement, and
2) cleanliness of surface.

Concrete overlays have performed successfully over the last 30 years with full, partial, or no bonding procedures. There are advantages and disadvantages for each alternative. A more detailed discussion can be found in reference 19.

3.1.5 Remaining Life

The proposed AASHTO procedure uses the remaining life concept to aid in the determination of the required overlay thickness. Few overlay design procedures utilize the remaining life concept into the design because an accurate method for measurement does not exist. Several alternatives were presented in the previous chapter for obtaining a representative R_{LX} value for the in situ pavement. Some of these procedures may provide a more accurate assessment than others, and are easier to implement than others. Agreement between the procedures is not consistent.

A choice must be made as to which procedure provides the most reliable value and can be determined with the most ease. AASHTO recommends using the NDT approach for damaged pavements. The cumulative traffic approach is similar to the approach used for determination of the R_{LY}, but reliable estimates of past traffic on secondary roads may be difficult to obtain. If continuous NDT data are available, relationships for the number of years vs. increased deflections could be utilized for better assessments of remaining life. The increase in deflection as the pavement deteriorates could be documented.

Another alternative is the continual monitoring of the condition of the pavement through condition surveys or serviceability ratings. Presently, condition surveys are performed by the maintenance department to ascertain which pavements are in need of rehabilitation. This information is available but may be in various forms due to differences in rating systems. This information would be much more beneficial if it was in a standardized format. Whichever method is chosen it should be used consistently on a particular type of roadway so a database can be created for better implementation in the future.

3.2 Required Inputs

The inputs needed to achieve these desired features for the AASHTO method are summarized in Table 3.2 and are discussed in the following sections.

Table 3.2. Needed Input Data.

Item	General Considerations
Deflection Data	1) equipment 2) frequency 3) seasonal variations
Estimation Layer Modulus	1) cores 2) regression equations 3) backcalculation
Traffic	1) method of collection 2) growth factors
Present Serviceability Rating	1) type of measurement 2) relationship to existing database

3.2.1 Deflection Data

Three general classes of NDT equipment are routinely used to collect deflection data for use in pavement evaluation and overlay design:

1) Static deflection equipment,

2) Steady-state dynamic deflection equipment, and

3) Impulse deflection equipment.

The basic characteristics and costs of each of the commonly used devices are summarized in Table 3.3. This discussion in this report is limited to the Dynaflect and the KUAB FWD, both of which are employed in Oregon.

A steady-state deflection device is capable of producing a sinusoidal vibration in the pavement with a dynamic force generator. The most commonly used device in Oregon is the Dynaflect. The Dynaflect is a trailer-mounted device that can be towed by a standard automobile. A static weight of 2000 to 2100 lbs (8896 to 9341 N) is applied to the pavement through a pair of rigid steel wheels. The dynamic generator uses a pair of unbalanced flywheels to produce a 1000-lb (4448 N) peak to peak force. The deflection is measured using five velocity transducers. The transducers are suspended from a bar which allows their placement at any location from the center point between the loading wheels to a distance of 5 ft (1.52 m) from that point. They are normally placed in the center at 1-ft (.305 m) intervals. The testing frequency and deflection measurements from all five transducers register simultaneously on the standard digital control. The normal sequence of operation is to move the device to the test point and lower the wheel and transducers. A test is run, and data collected. If the next test point is nearby, the sensors can be raised and the device can be moved to the next site

Table 3.3. Characteristics of Commercially Available NDT Devices in 1984 (41).

Device Name	Principal of Operation	Load Actuator System	Min. Load	Max. Load	Static Weight on Plate	Type of Load Transmission	Method of Recording Data
Benkelman Beam (AASHTO)	Deflection beam	Loaded truck axle	N/A	N/A	N/A	Truck wheels	Manual
Deflection Beam (British)	Deflection beam	Loaded truck axle	N/A	N/A	N/A	Truck wheels	Manual
La Croix Deflectograph	Mechanized deflector beam	Moving truck loaded with blocks or water	Empty truck weight	Loaded truck weight	N/A	Truck wheels	Manual, printer, or automated
Dynaflect	Steady state vibratory	Counter rotating masses	1,000	1,000	2,100	Two 16' dia. urethane coated steel wheels	Manual, printer, or automated
Model 400 B Road Rater	Steady state vibratory	Hydraulic actuated masses	500	2,800	2,400	Two 4x7 pads with 5.5' center gap***	Manual, printer, or automated
Model 2000 Road Rater	Same	Same	1,000	5,500	3,800	Circular plate 18' dia.**	Same
Model 2008 Road Rater	Same	Same	1,000	8,000	5,800	Same	Same
KUAB 50 Falling Weight Deflectometer	Impulse	Two dropping masses	1,500	12,000	?	Sectionalized circular plate 11.8' dia.*	Manual, printer, or automated
KUAB 150 Falling Weight Deflectometer	Same	Same	1,500	35,000	?	Same	Same
Dynatest Model 800 Falling Weight Deflectometer	Impulse	Dropping masses	1,500	24,000	?	Circular plate 11.8' dia.	Manual, printer, or automated

*Solid plates and plates of other diameters are available.
**Plates of other diameters are available.
***Circular plates are available.

Table 3.3. Characteristics of Commercially Available NDT Devices in 1984 (41) (continued).

Device Name	Type of Carriage	Type of Prime Mover	Basic Cost	Contact Area	Vibratory Frequency & Range	Deflection Measuring System	Number of Deflection Sensors	Normal Spacing of Sensors	Load Measuring System
Benkelman Beam (AASHTO)	N/A	N/A	$ 1,000	N/A	N/A	Dial indicator	1	N/A	None
Deflection Beam (British)	N/A	N/A	$ 1,500	N/A	N/A	Dial indicator	1	N/A	None
La Croix Deflectograph	Truck	None	$166,500**	N/A	N/A	Inductive displacement transducers	2 (one in each wheel path)	N/A	None
Dynaflect	Trailer	Tow vehicle	$ 22,185	32 in.^{2}	8 Hz	Velocity transducers	5	Center and at 1" intervals	None
Model 400 B Road Rater	Trailer*	Tow vehicle	$ 30,580	56 in.^{2}	5 Hz to 70 Hz	Velocity transducers	4	Center and at 1" intervals	Load cell
Model 2000 Road Rater	Same	Same	$ 40,800	254 in.^{2}	Same	Same	4	Same	Same
Model 2008 Road Rater	Same	Same	$ 64,000	254 in.^{2}	Same	Same	4	Same	Same
KUAB 50 Falling Weight Deflectometer	Trailer	Tow vehicle	$ 70,000	109 in.^{2}	N/A	Seismic deflection transducers	5	Center and 0.6' to 8.0'	Load cell
KUAB 150 Falling Weight Deflectometer	Same	Same	$ 85,000	109 in.^{2}	N/A	Same	5	Same	Same
Dynatest Model 8000 Falling Weight Deflectometer	Trailer	Tow vehicle	$ 86,500	109 in.^{2}	N/A	Velocity transducers	7	Center and 0.6' to 7.4'	Load cell

*Earlier versions of the Model 400 were mounted on vehicles.
**$71,000 without truck, but requires 1 to 3 man-months to install on purchaser's vehicle.

on the testing wheels at speeds of up to 6 mph (9.65 km/hr). Technical limitations include:

1) A peak to peak force of only 1000 lbs (453 kg).

2) The load cannot be varied and frequency may not be changed.

3) The deflection directly under the load cannot be measured.

4) It is difficult to determine the contact area.

Impulse deflection equipment deliver a transient impulse force to the pavement surface. Force impulses are normally generated by dropping a weight from a certain height onto an impact plate which has been placed on the pavement surface. By varying the magnitude of the falling mass and the drop height the impulse force can be varied. Advantages of impulse devices include an accurate measurement of the deflection basin, and relatively small preloads compared to the actual loads. Also, loadings in the range of actual truck wheel loads can be applied and the resulting deflections are similar to moving wheel loads. Three models are currently in use in the United States and Europe. This discussion is limited to the KUAB FWD, which is employed in the state of Oregon.

The KUAB FWD is a trailer-mounted device that can be towed by a standard size automobile. The impulse force is created by dropping a set of two weights from different heights. By varying the drop heights and weights the force can be varied from 2698 to 35,000 lbs (12.0 to 155.7 kN). The two-mass falling weight system is used to create a smooth rise of the pulse on pavements with both stiff and soft subgrade support. A rise time from no load to peak load is developed in 28 milliseconds, which approximates the load development time of a vehicle traveling at approximately 44 mph (70.8 km/hr). The load is transmitted to the pavement through an 11.8 in. (30 cm) diameter loading plate. On smooth pavements, a solid plate is recommended. On uneven surfaces, a segmented steel plate with hydraulic load distributions should be used. The deflection is measured using five absolute seismic displacement transducers (seismometers) that are lowered automatically with the loading plate. One sensor is placed through the middle of the loading plate and the remaining sensors can be placed from 7.9 to 100 in. (20.1 to 254 cm) from the center of the plate. The signals from the seismic transducers and load cell are fed into a computer* which also controls the complete operation of the device.

The frequency of the tests is also important. Currently, ODOT chooses a 1000-ft (304.8 m) representative section in a project mile and takes deflection readings every 50 ft (15.24 m). Other agencies, such as the Asphalt Institute, recommend 250-ft (76.2 m) intervals for deflection testing.

Currently, Oregon is also involved in the determination of seasonal deflection values. Relationships are being established for the time of year vs. the change in deflection value for various locations. Examples of these plots are given in Figure 3.1. At this time, the previous 5-day precipitation values are more important than the season of the year for determining the variation in deflection values (33). These relationships can provide information about which critical period of the year should deflection measurements be taken.

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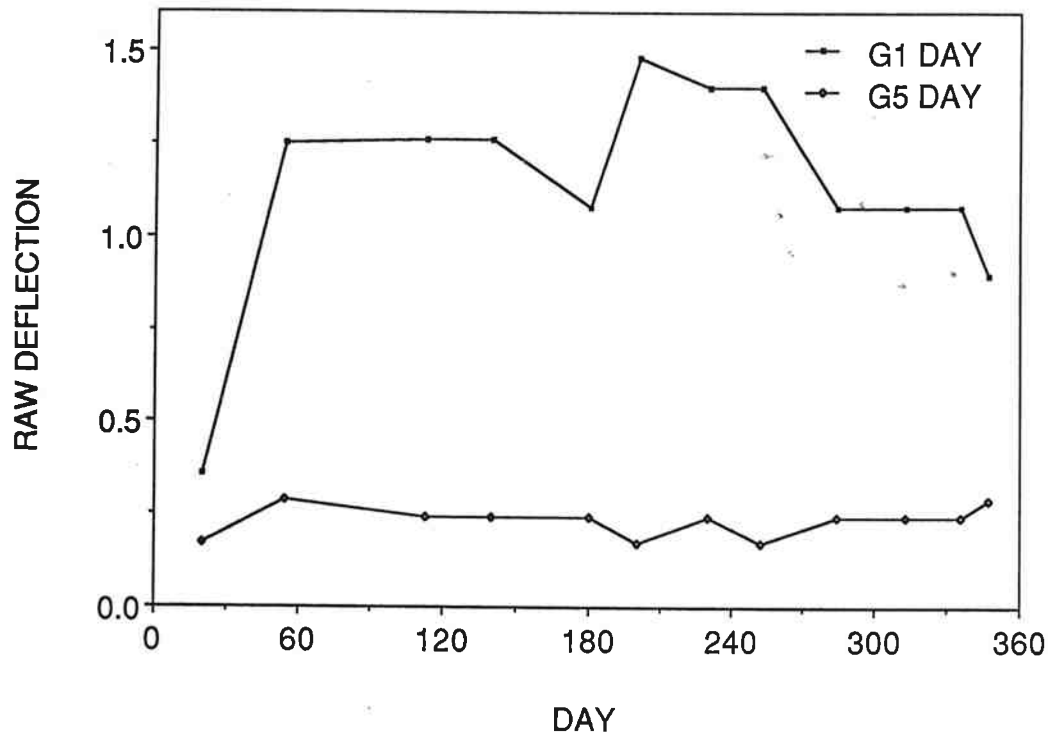
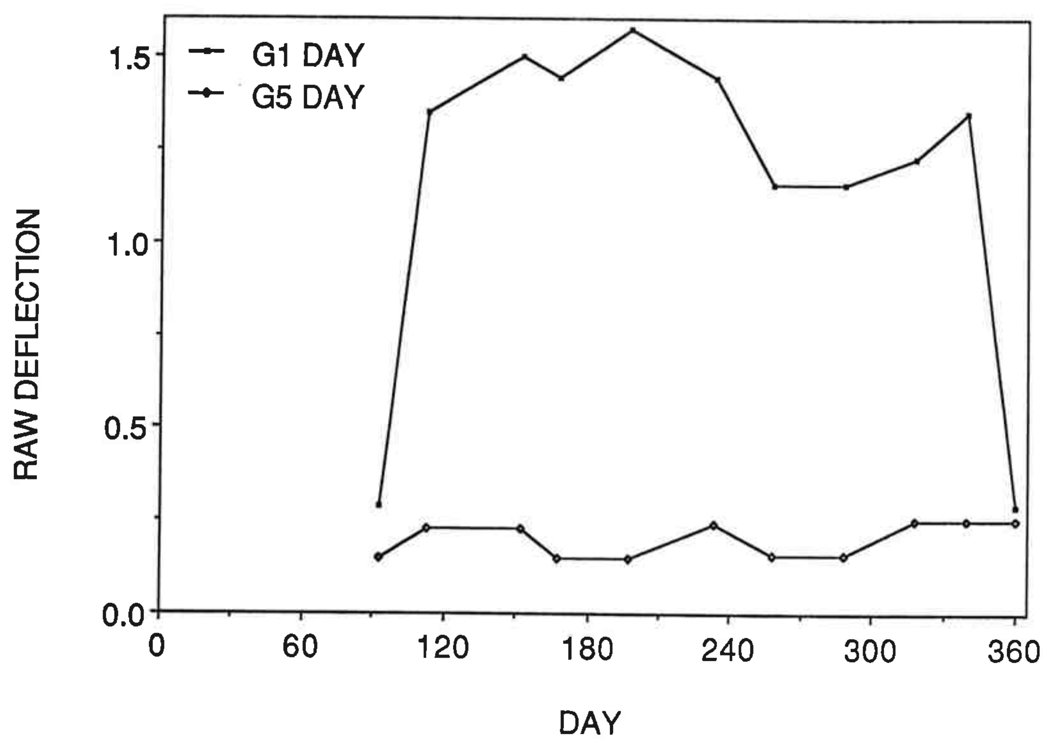
*HP-85 or COMPAQ's.

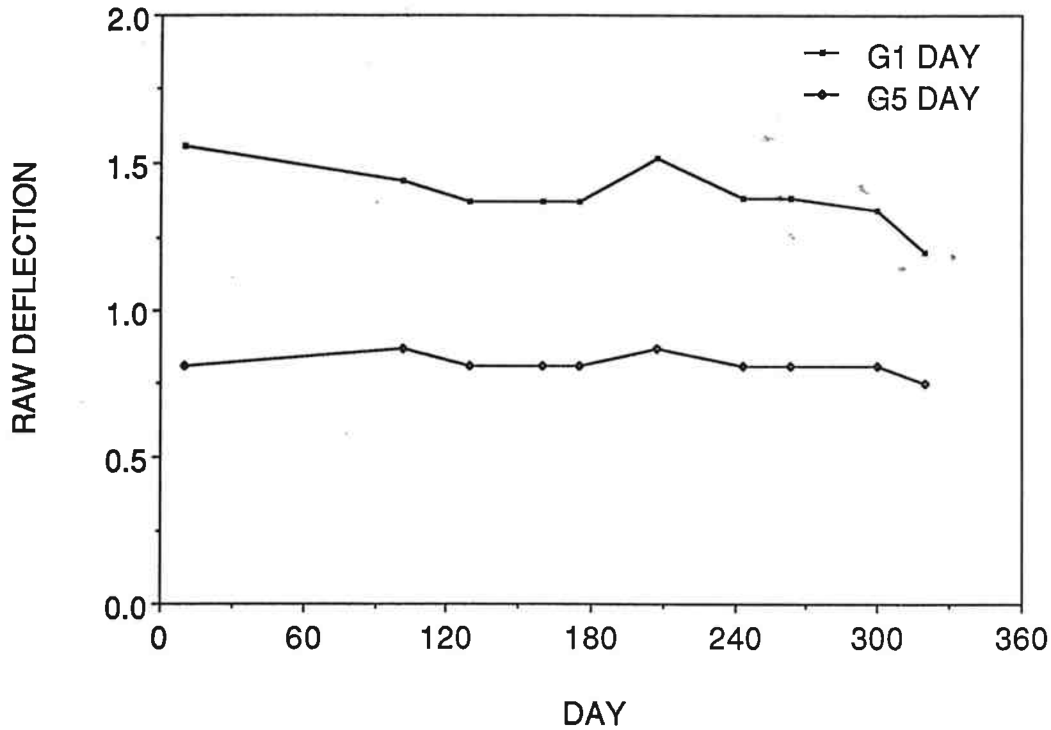


a) Raw Deflection vs. Julian Day for Geophone 1 (G1) for a Typical Eastern Site

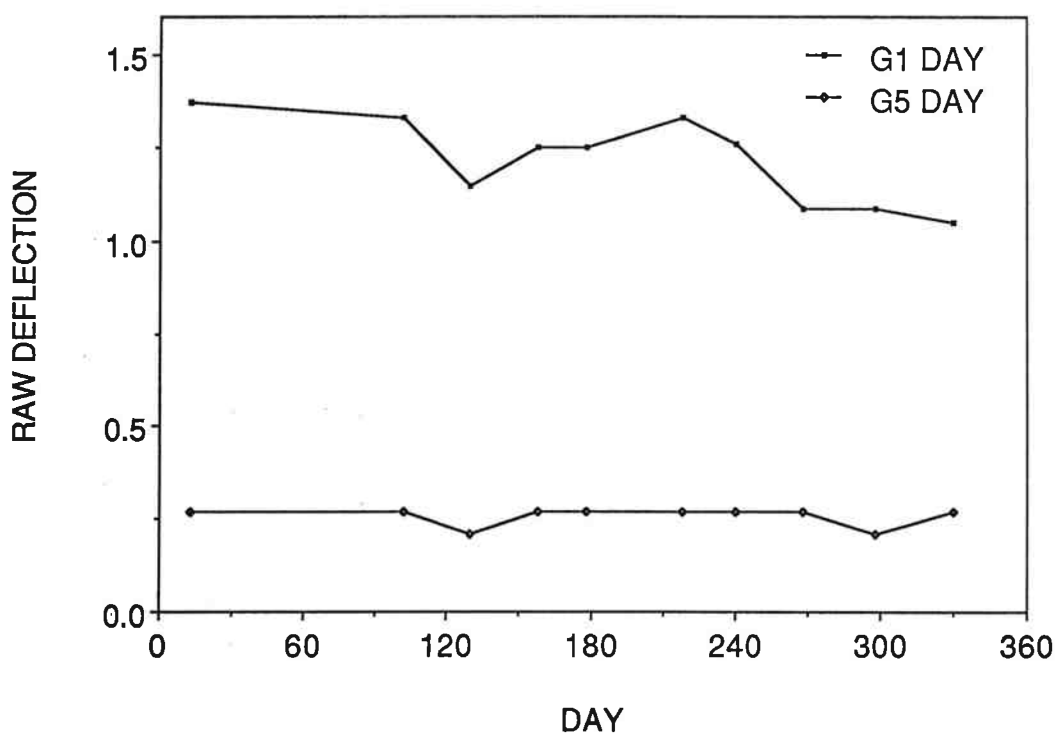


b) Raw Deflection vs. Julian Day for Geophone 1 (G1) for a Typical Cascade Site

Figure 3.1. Deflection Variation During the Year (33).



c) Raw Deflection vs. Julian Day for Geophone 1 (G1) for a Typical Coastal Site



d) Raw Deflection vs. Julian Day for Geophone 1 (G1) for a Typical Valley Site

Figure 3.1. Deflection Variation During the Year (33) (cont.).

3.2.2 Estimating Layer Modulus

The predicted modulus values for the untreated bases and subgrade layers should be more reliable and involve less computer time if cores are taken on the project sites and their moduli obtained from the laboratory. This suggestion requires the purchase of resilient modulus testing apparatus and proper training of personnel. Oregon DOT is currently in the process of purchasing such equipment.

The amount of cores taken should depend upon variation in surfacing materials along the project. Cores should be taken where the pavement surface varies from preceding test sections. For the preliminary investigations, three cores were taken on each of the three projects for the 1000-ft (304.8 m) section. One core was taken at the beginning, middle, and end of the section.

Regression equations have been developed by several agencies to assist in determining the subgrade modulus in a pavement system (see Table 3.4). The alternative to using regression equations is calculating layer moduli using backcalculation techniques, such as BISDEF, ELSDEF and MODCOMP2 (39). This discussion is limited to BISDEF, which was developed by the U.S. Army Corps of Engineers, Waterways Experiment Station (8). It uses a deflection basin from nondestructive testing results to predict the elastic moduli of several layers. This is accomplished by matching the calculated deflection basin to the measured deflection basin.

The basic assumption of BISDEF is that dynamic field deflections can be reproduced using layered elastic theory. This method uses the BISAR program to compute deflections, stresses, and strains under investigation. To determine layer moduli, the basic inputs for analysis include the elastic

Table 3.4. Various Regression Equations for Subgrade Modulus.

AASHTO (2)	$E_{SG} = PS_f/d_r r$	P - load (lbs) S_f - modulus prediction factor r - distance to geophone d_r - deflection at distance r
Ullidtz (44)	$E_{SG} = \dfrac{(1-u_i^2)P_a{}^2}{d_2 r}$	u_i - Poisson's ratio P - contact stress a - radius plate r - distance to geophone d_r - deflection at distance r
Washington (28)	$E_{SG} = 7.61(P/D_3)$ $R^2 = .98$	P - load (lbs)
	$E_{SG} = 5.77(P/D_4)$ $R^2 = .99$	D_x - deflection mils at x feet from geophone

layer pavement characteristics as well as deflection basin values. The inputs for each layer are:

1) Poisson's ratio,
2) thickness of each layer,
3) range of allowable modulus,
4) initial estimate of modulus,
5) deflections at a number of sensor locations, and
6) maximum acceptable error in deflection.

The modulus of any surface layer may be assigned or computed. If assigned, the value will be based on the type of material or properties of the material at the time of testing. The number of layers with unknown modulus values cannot exceed the number of measured deflections. Best results are obtained when not more than three layers are allowed to vary. The Army Corps of Engineers suggests placement of a rigid layer at 20 ft (6.1 m) from the pavement surface to improve the matching of measured to calculated deflection basin. The program, using an iterative process, provides the best fit between measured and computed deflection values, one that minimizes the error sum between the computed deflections and measured deflections (39).

All of the backcalculating programs can be supplemented with laboratory determined modulus for bond materials such as asphalt concrete, portland cement concrete, and cement-treated bases. If these values can be determined accurately in the laboratory, then the values may be substituted into the backcalculating programs.

3.2.3 Traffic Data

Traffic loading directly influences the recommended thickness for an overlay design. For example, Packard (32) compared two different methods of evaluating mixed traffic and illustrated the potential differences in the final design. Table 3.5 and Figure 3.2 show how the two different approaches handle the damage contribution of mixed traffic. Since PCA and AASHTO evaluate relative damage for traffic differently, the thickness recommendations for an overlay may vary. The huge numbers of low-weight axle loads in the AASHTO evaluation become the governing criteria for thickness as opposed to the PCA viewpoint (heavy weight axles influence pavement performance and design). Further, many agencies have a unique set of equivalency factors as shown in Table 3.6.

Recently, weigh-in-motion data have been collected in Oregon which breaks down the five categories Oregon currently uses into 19 vehicle classifications (Figure 3.3); AASHTO, on the other hand, recommends 13 vehicle classifications in the new 1986 AASHTO guide as shown in Table 3.7. Although breaking traffic down into several categories for analysis reveals a more accurate assessment of the pavement damage, most agencies are not equipped well enough to predict this type of traffic at the present. The 1986 AASHTO procedure requires cumulative traffic estimates on highways to determine remaining life. This information can generally be determined for Interstates, but is not available for many secondary roads. The designer has a few options in estimating traffic under these circumstances (34):

1) Federal roads have traffic counts available for every two years, and state roads also have yearly counts. Neither break down traffic by axle type, but an average percentage of each for that type of road could be estimated and applied to the total count.

Table 3.5. Comparison of Damage Effects for Mixed Traffic (32).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
			AASHTO Design		Evaluation of Relative Damage		
Axle Load, kips	Axle Load Repetitions	PCA Design[1] % Damage	Equiv. Factor	ESAL[2]	PCA, %[3]	AASHTO, %[4]	Effect at Road Test
30S	6,570	9.46	7.79	51,180	72	4	Substantial
22.4S	114,600	2.20	2.48	284,208	17	23	Some
18S	367,190	1.53	1.00	367,190	11	29	Very slight
12S	2,660,000	0	0.18	478,800	0	39	None to very slight
6S	5,582,000	0	0.01	55,820	0	5	None (assumed)
	Total	13.19	-	1,237,198	100	100	-

Conditions: 8-in. slab, PCA Axle Load Category 3, dowels, no concrete shoulder, S = single axle.

[1]Fatigue or erosion damage, whichever is greater, from computer program, PCAPAV
[2]Col. 2 x Col. 4
[3]100 x Col. 3/Total Col. 3
[4]100 x Col. 5/Total Col. 5

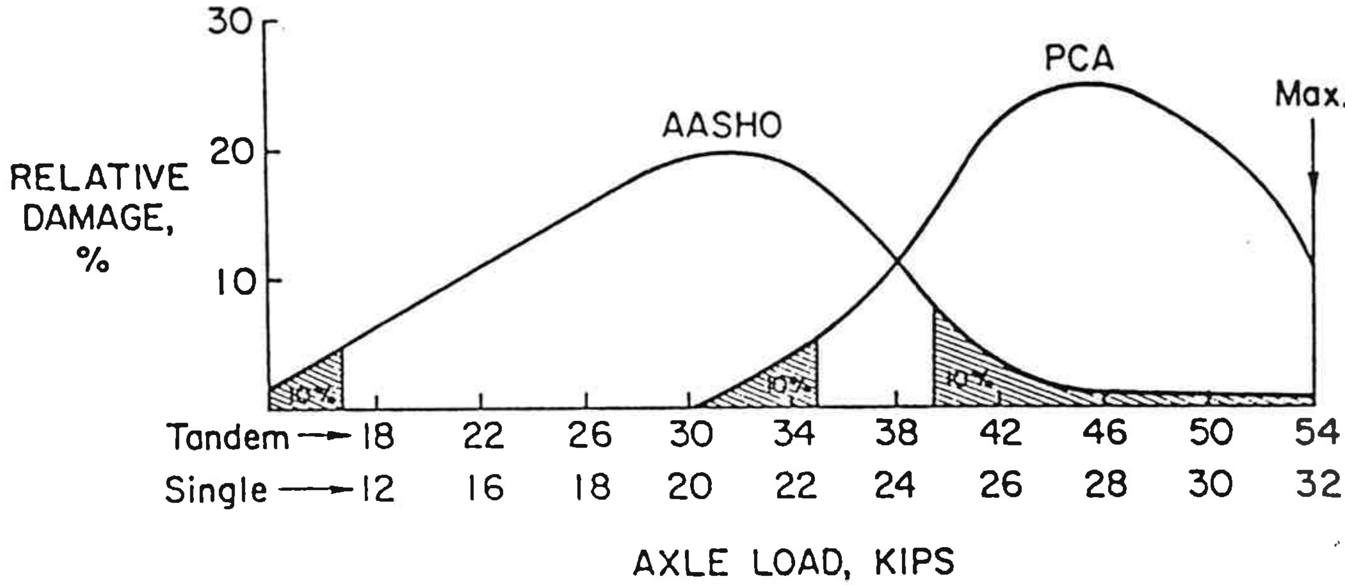


Figure 3.2. Effects of Axle Load Magnitudes, AASHTO E-18's vs. PCA Design (32) (Computed from a N.Y. W-4 Table).

Table 3.6. Conversions of Axles to 18 kip Equivalent Single Axle Load (ESAL) for Rigid Pavements.

Agency	Axles (kips)	Conversion Constant to 18^k ESAL
Oregon DOT (31)	2	.1096
	3	.274
	4	.315
	5	1.014
	6	.8904
Caltrans (9)	2	.1889
	3	.5041
	4	.8055
	5	1.8870
Washington DOT (28)	2	.4- .44
	3	.6- .66
	4	.8- .88
	5	1.0-1.10
Weigh-in-Motion Station in Oregon (23)	2	.12
	3 single units	.61
	3 combinations	.43
	4 single units	.40
	5 series	1.55
	5 twins	2.00

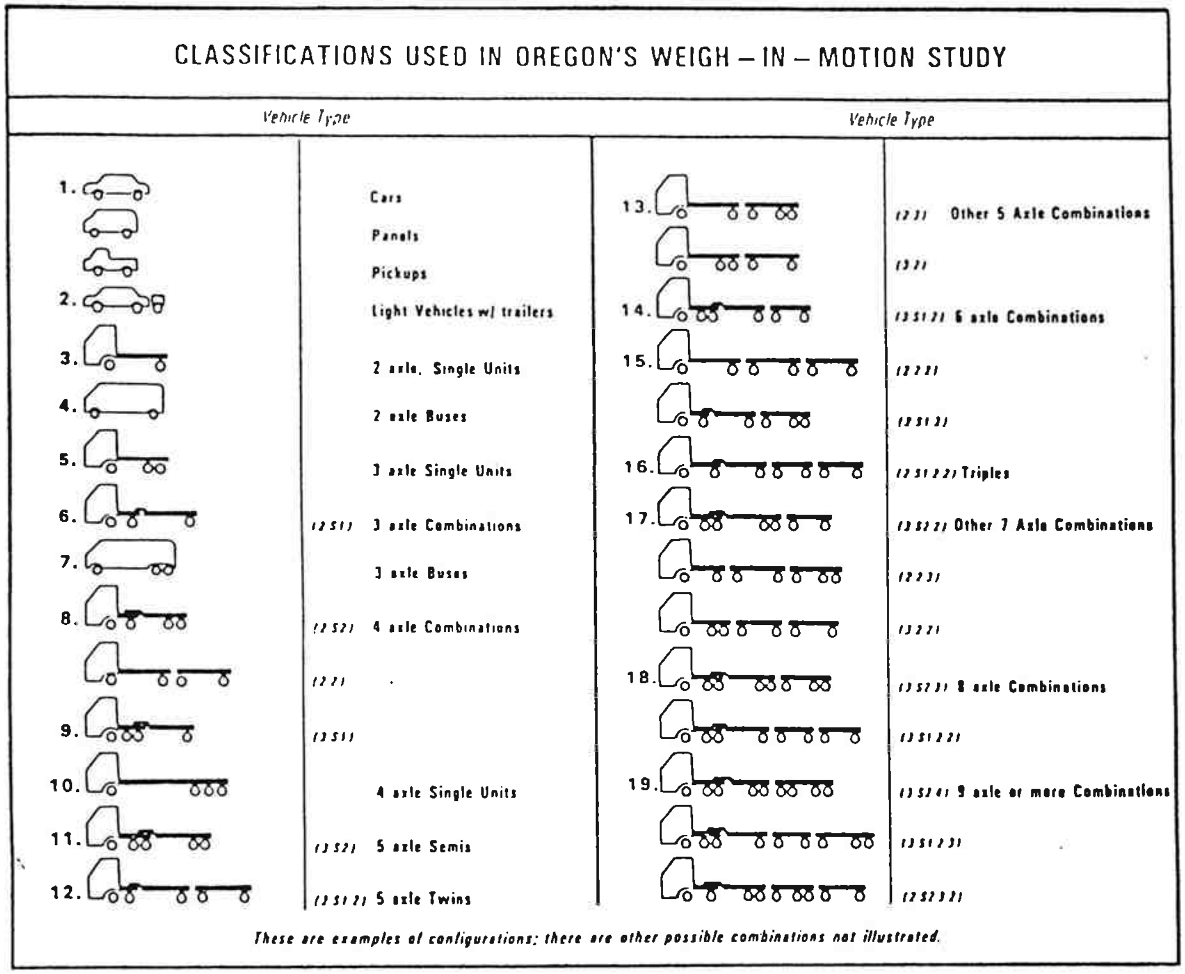


Figure 3.3. Classifications Used in Oregon's Weigh-in-Motion Study (23).

Table 3.7. Worksheet for Calculating 18 kip Equivalent Single Axle Load (ESAL) Applications (2).

Analysis Period = ___ Years

Location: ______________________ Assumed SN or D = ___

Vehicle Types	Current Traffic (A)	Growth Factors (B)	Design Traffic (C)	ESAL Factor (D)	Design ESAL (E)
Passenger Cars					
Buses					
Panel and Pickup Trucks					
Other 2-Axle/4-Tire Trucks					
2-Axle/6-Tire Trucks					
3 or More Axle Trucks					
All Single Unit Trucks					
3-Axle Tractor Semi-Trailers					
4-Axle Tractor Semi-Trailers					
5+-Axle Tractor Semi-Trailers					
All Tractor Semi-Trailers					
5-Axle Double Trailers					
6+-Axle Double Trailers					
All Double Trailer Combos.					
3-Axle Truck-Trailers					
4-Axle Truck-Trailers					
5+-Axle Truck-Trailers					
All Truck-Trailer Combos.					
All Vehicles				Design ESAL	

2) Trends for stable environments could be applied to similar roads, but many roads are seasonal.

3) Planning agencies in larger cities have models for estimates of traffic.

3.2.4 Present Serviceability Rating (PSR)

PSR values are a measure of the ride judged by a panel on a scale of one to five with 1 being poor and 5 being excellent. Assigning PSR values to pavements is in the preliminary stages in Oregon. At this time the state has not collected enough data to obtain reliable PSR rating values, so visual condition factor, rather than its ride, are used to assess the performance of the road. The condition factors assigned are from 1 to 5, with a rating of 5 being poor and 1 being excellent. The state has attempted to correlate this with PSI values (30): however, at present there are no definite plans to change to the PSI concept.

3.3 AASHTO Framework

As previously discussed, four types of overlay situations are considered in the 1986 AASHTO Guides:

1) Flexible overlay on existing flexible pavement.

2) Flexible overlay on existing rigid pavement.

3) Rigid overlay on existing rigid pavement.

4) Rigid overlay on existing flexible pavement.

The steps outlined in Chapter 2 are necessary for all overlay situations (see Figure 3.4). Data requirements necessary for each procedure are given in Table 3.8. Each procedure is described below step by step. Worked examples are given in Appendix D.

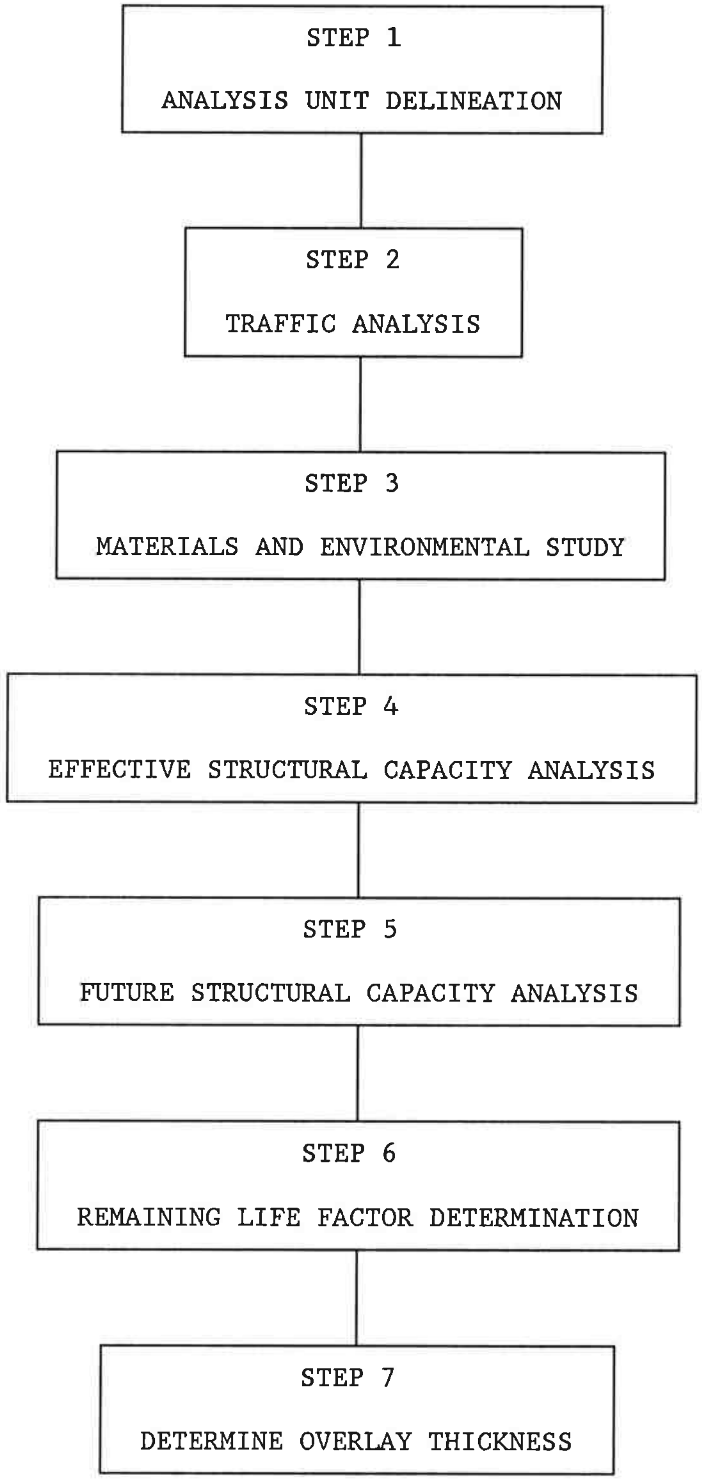


Figure 3.4. Steps Necessary for AASHTO Implementation.

Table 3.8. Data Requirements for Overlay Design.
(FWD, Dynaflect, Benkelman Beam)

Material Types	Data Requirements
1) AC/AC	radius of loading plate estimate equivalent thickness of pavement structure Poisson's ratio for all layers distance of geophone from center load dynamic or impulse load deflection measurements (basin or max) Resilient moduli (backcalculation or laboratory testing) thickness AC + thickness base reliability standard deviation estimate future traffic present P_t and at end of service P_{t2} accumulative traffic or original SN
2) PCC/AC	mean pavement temperature C_d value for particular structure unadjusted max deflection NDT load plate diameter dynamic load
3) AC/PCC	slab length } to check for min. thickness mean annual temp. } thickness PCC thickness subbase P_{t2} (at end of overlay) backcalculation* or visual condition rating with modulus testing of E_{SB}, E_{SG} reliability standard deviation accumulated traffic* or P_{t1} (at time of overlay) estimated future traffic
4) PCC/PCC	Design as new pavement

*Preferred

3.3.1 Type 1 Flexible Overlay on Flexible Pavements

The required thickness of an overlay is determined as follows:

$$h_{OL} = SN_{OL}/a_i = (SN_y - F_{RL}SN_{xeff})/a_i \tag{3.2}$$

where:

SN_y = structural number needed for future traffic loads,

SN_{xeff} = effective structural number of in situ pavement,

F_{RL} = remaining life factor,

h_{OL} = thickness of the overlay (in.),

a_i = structural layer coefficient, and

SN_{OL} = structural number of required overlay.

A flow chart outlines the procedure in Figure 3.5. The analysis unit is established from Step 1. The traffic analysis, Step 2, is then performed. The future traffic "y" is determined for use in Steps 5 and 6. If the historic traffic information "x" is not available, one of the other methods discussed in Step 6 is employed to determine the remaining life. The type of NDT device employed will influence the type of information obtained in Step 3. If the NDT deflection method 1 is used, the effective moduli of all pavement layers as well as the existing subgrade modulus are computed from a backcalculation technique using the measured deflection basin. The existing subgrade modulus is a necessary input for Step 5. The individual structural layer coefficients (a_i) are estimated from the layer modulus. The results of the pavement layer moduli predictions are used in Step 4. For NDT Method 1, SN_{xeff} is found from:

$$SN_{xeff} = \Sigma\, a_i h_i \tag{3.3}$$

where:

Flowchart AC/AC

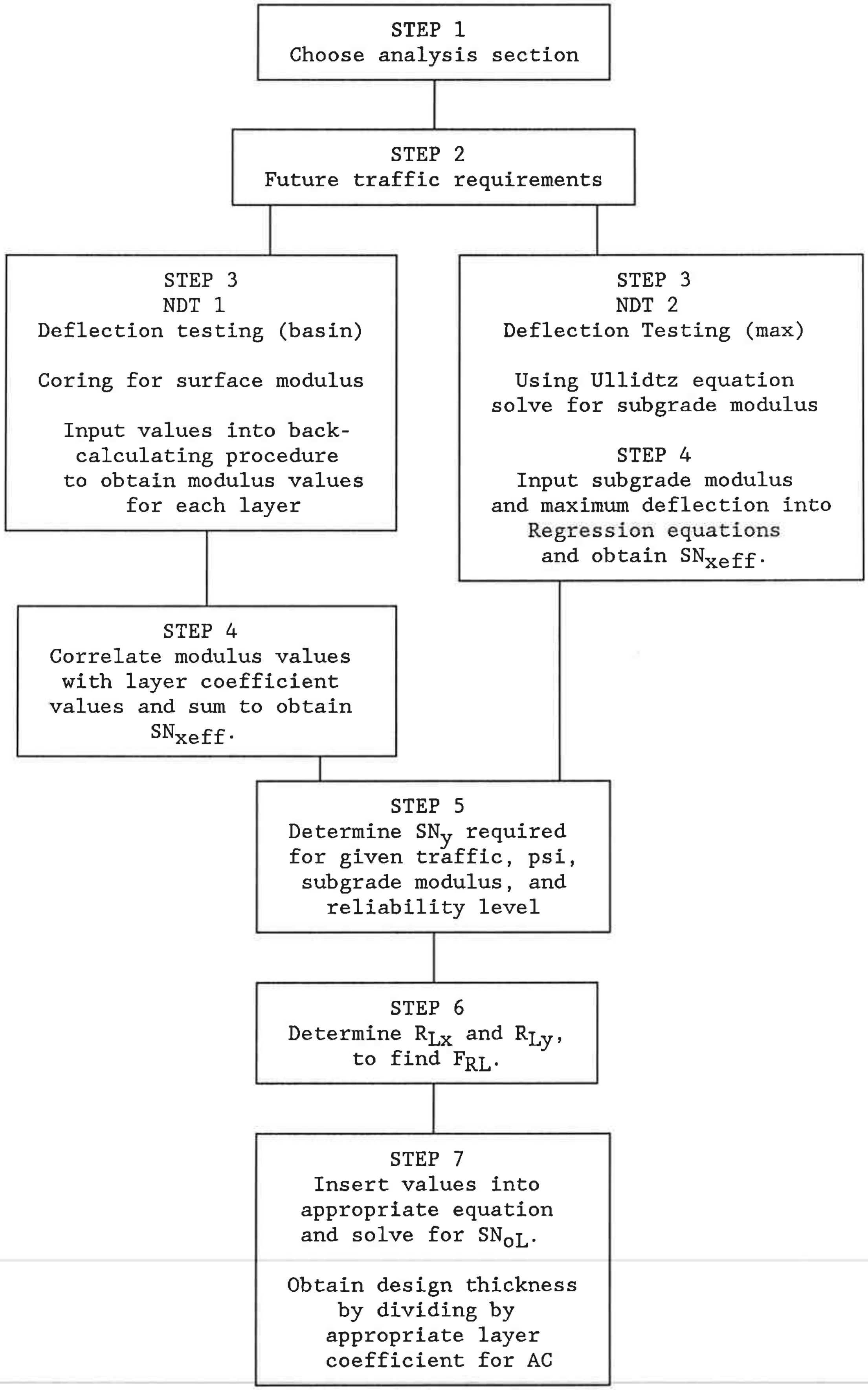


Figure 3.5. Flowchart of Flexible Overlays Over Flexible Pavements.

a_i = layer coefficient, and

h_i = thickness of layer.

Step 5 involves the determination of the future required SN_y from the design subgrade modulus (Step 3), future design traffic repetitions, "y", and the designer decision regarding the terminal serviceability (P_{t2}) at the end of the overlay period. For Step 6, F_{RL} is determined from one of the five approaches preferably the NDT Approach or the Traffic Approach. Once this value is known the overlay equation can be used directly to compute h_{OL}, knowing SN_y, F_{RL}, SN_{xeff}, and a_i.

3.3.2 Type 2 Flexible Overlay on Rigid Pavements

This type of overlay situation is not often used in Oregon. In this situation, once the PCC pavement is cracked, it is no longer in a rigid condition. Even after the overlay is placed, the cracking of the PCC layer may continue causing the overlain pavement to approach a more flexible condition with time and traffic. Problems arise in the determination of the equivalent structural capacity (whether D_o or SN_o should be used).

Determination of the structural overlay is only one of the two concerns. The other factor which should be considered is the reflective cracking potential of the asphalt overlay. Reflective cracking can be minimized by the use of thicker AC overlays, the break seat approach, saw cutting matching transverse joints in overlay, use of crack relief layers, stress absorbing membrane interlayers, and fabric/membrane interlayers. However, none of these methods currently guarantee the elimination of reflective cracking.

Different forms of the equation in Table 3.9 are used depending upon whether a normal structural overlay analysis or break seat overlay approach is contemplated and which analysis procedure is used to determine the overlay

Table 3.9. Summary of Overlay Equations Used in Flexible Overlay Over Existing Rigid Pavement Analysis (2).

Major Overlay Condition	Specific Method Used	SN_{OL} Equation
Normal Structural Overlay	NDT Method 1	$SN_{OL} = SN_y - F_{RL}\ (0.8\ D_{xeff} + SN_{xeff\text{-}rp})$
	NDT Method 2	$SN_{OL} = SN_y - F_{RL}\ SN_{xeff}$
	Visual Condition Factor	$SN_{OL} = SN_y - F_{RL}\ (a_{2r}D_o + SN_{xeff\text{-}rp})$
Break-Seat Overlay	Estimating Nominal Crack Spacing	$SN_{OL} = SN_y - 0.7(0.4D_o + SN_{xeff\text{-}rp})$*
	Post Cracking NDT	
	a. NDT Method 1	$SN_{OL} = SN_y - 0.7(a_{bs}D_o + SN_{xeff\text{-}rp})$
	b. NDT Method 2	$SN_{OL} = SN_y - 0.7\ SN_{xeff}$

*Special Note: The coefficient of D_o (i.e. 0.4) actually varies from 0.35 for a nominal crack spacing of approximately 2.0 ft to a value of 0.45 for a nominal crack spacing of approximately 3.0 ft.

within each of the two major categories. Table 3.9 summarizes the specific equations to be used for the two major categories. The variables are defined below:

D_o = Existing PCC layer thickness.

D_{xeff} = Effective thickness of the in situ PCC layer reflecting its reduced modulus value.

SN_{xeff} = Total effective structural number of the existing pavement structure above the subgrade.

$SN_{xeff-rp}$ = Effective structural capacity of all remaining layers above the subgrade except for existing PCC layer.

a_{2r} = The structural layer coefficient of the existing cracked PCC pavement layer. This value is used in a normal structural overlay analysis and has been related to the value of the visual condition factor C_v.

a_{bs} = The structural layer coefficient of the PCC pavement layer after it has been broken during the break seat approach. This value is related to the in situ (broken) PCC modulus.

The required thickness of the overlay is determined from the equation

$$h_{OL} = SN_{OL}/a_i \tag{3.4}$$

where:

h_{OL} = thickness overlay

SN_{OL} = structural number required for overlay, and

a_i = structural layer coefficient.

The general methodology follows the same six steps previously mentioned. Figure 3.6 shows a flow chart of these steps.

Once the structural overlay thickness is determined it must be compared with the minimum thickness to prevent reflective cracking. The minimum thicknesses developed by the Asphalt Institute are shown on Table 3.10.

3.3.3 Type 3 Rigid Overlay on Rigid Pavement

In this type of overlay, three potential bonding situations can exist: (a) full bond, (b) partial bond, and (c) unbonded. Table 3.11 summarizes portland cement concrete overlay criteria for existing rigid pavements. The seven steps can be implemented easily as shown in Figure 3.7. The analysis unit and traffic study is performed for Steps 1 and 2. If a full bond or a partial bond is being considered, accurate historic traffic data are essential for determining the existing pavement life. The in situ effective moduli of the PCC layer can be directly inserted into Step 4, while the subbase/subgrade moduli will be used to determine the subgrade reaction k_c necessary for Step 5 (which is outlined in AASHTO guide Part II). The effective PCC thickness is calculated knowing the PCC modulus developed in Step 3. If NDT derived data are unavailable, one of the following alternate procedures may be used to estimate D_{xeff}: (a) Visual Condition Factor, (b) Nominal Size of Slab Fragments, or (c) Remaining Life Approach.

Five approaches are available for determining R_{LX}. Once the R_{LX} and R_{LY} are established, F_{RL} can be determined. The overlay equation can then be selected from Table 3.9 and the rigid overlay may be completed.

Flowchart AC/PCC

STEP 1
Choose analysis section

STEP 2
Determine traffic requirements

STEP 3
NDT 1
Deflection testing (basin)

Test cores for surface modulus

Input values into backcalculating procedure to obtain modulus values for each layer

STEP 3
NDT 2
Deflection testing (max)

Use Ullidtz equation to solve for subgrade modulus

STEP 4
Input values into regression equations and obtain SN_{xeff}.

STEP 4
with layer coefficient values and sum to obtain $SN_{xeff\text{-}rp}$.

Use cracked PCC layer coefficient times the slab thickness (D_o)

Use .8 times D_{xeff} (determined from knowing E_{pcc} and D_o)

Use Table 3.9 to obtain SN_{xeff}.

STEP 5
Determine SN_y from new design

STEP 6
Determine R_{Lx}, R_{Ly}, and find F_{RL}.

STEP 7
Input values into equations and solve for SN_{oL}

Obtain design thickness by dividing by appropriate layer coefficient for AC

Figure 3.6. Flowchart of Flexible Overlays Over Portland Cement Concrete Pavements.

Table 3.10. Minimum Asphalt Concrete Structural Overlay Thickness for PCC Pavement (from The Asphalt Institute MS-17) (2).

Existing PCC Length (ft)	h_{o1} (min - in.) Maximum Annual Temperature Differential (°F)					
	30	40	50	60	70	80
10	4	4	4	4	4	4
15	4	4	4	4	4	4
20	4	4	4	4	5	5.5
25	4	4	4	5	6	7
30	4	4	5	6	7	8
35	4	4.5	6	7	8.5	*
40	4	5.5	7	8	*	*
45	4.5	6	7.5	9	*	*
50	5	7	8.5	*	*	*
60	6	8	*	*	*	*

*Alternative other than thickness of AC overlay should definitely be considered to minimize reflective cracking.

Table 3.11. Summary of Concrete Overlays on Existing Concrete Pavements (2).

		Unbonded or Separated Overlay	Partially Bonded or Direct Overlay	Bonded or Monolithic Overlay
Type of Overlay		T_R T_O	T_R T_O	T_R T_O
Procedure		Clean surface debris and excess joint seal, place separation course, place overlay concrete.	Clean surface debris and excess joint seal and remove excessive oil and rubber - place overlay concrete.	Scarify all loose concrete, clean and acid etch surface - place bonding grout and overlay concrete.
Matching of Joints in Overlay and Pavement	Location	Not necessary	Required	Required
	Type	Not necessary	Not necessary	Required
Reflection of Underlying Cracks to be Expected		Not normally	Usually	Yes
Requirement for Steel Reinforcement		Requirement is independent of the steel in existing pavement or condition of existing pavement.	Requirement is independent of the steel in existing pavement. Steel may be used to control cracking which may be caused by limited nonstructural defects in pavement.	Normally not used in thin overlays. In thicker overlay, steel may be used to supplement steel in existing pavement.
T_R Should be Based on the Flexural Strength of		Overlay concrete	Overlay concrete	Existing concrete
Minimum Thickness		6 inches	5 inches	1 inch
Applicability of Various Overlay Types				
Structural Condition of Existing Pavement				
No Struct. Defects C = 1.0*		Yes	Yes	Yes
Limited Struct. Defects C = 0.75*		Yes	Only if defects can be repaired	Only if defects can be repaired
Severe Struct. Defects C = 0.35*		Yes	No	No
Surface Cracks, Scaling, Spalling, and Shrinkage Cracks				
Negligible		Yes	Yes	Yes
Limited		Yes	Yes	Yes
Extensive		Yes	No	Yes

*C values apply to structural conditions only, and should not be influenced by surface defects.

Flowchart PCC/PCC

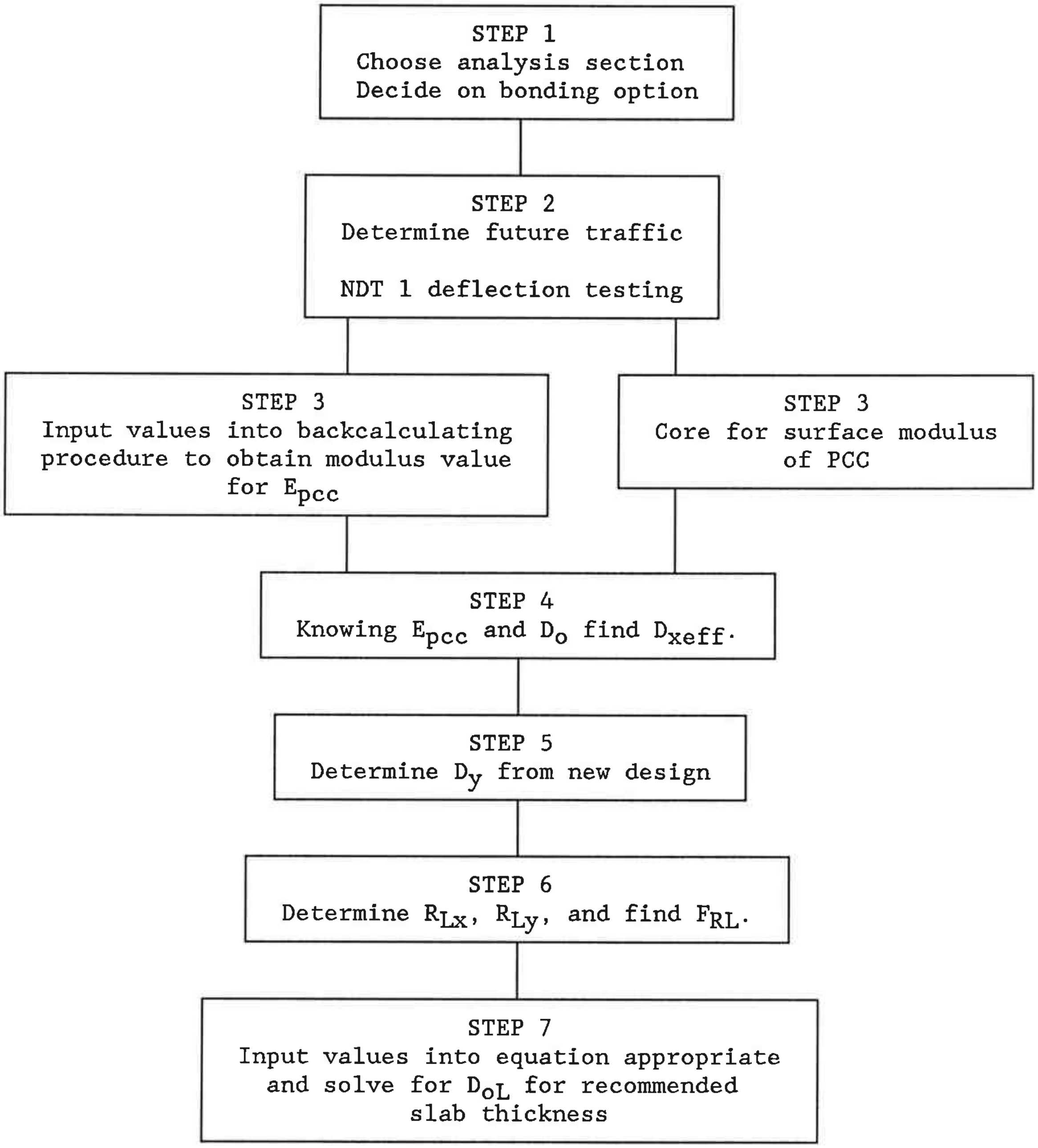


Figure 3.7. Flowchart for Portland Cement Concrete Overlay Over Portland Cement Concrete Pavement.

3.3.4 Type 4 Rigid Overlay on Flexible Pavement

Rigid overlays are being used more and more on flexible pavements in Oregon. In most cases, a leveling course or surface milling to correct surface irregularities is often necessary before a rigid overlay may be constructed. In Oregon, the mill depth could be as great as 10 to 12 in. The remaining pavement is considered as the composite foundation support for the rigid overlay. The design analysis consists of determining the composite modulus of subgrade reaction, k, for the existing pavement. This information is then used to design a new rigid pavement (Figure 3.8). To evaluate the composite k value for the existing pavement, the maximum NDT deflection under the load plate determined as follows:

$$d_{oc} = d_o k_d \tag{3.5}$$

where:

d_{oc} = adjusted deflection,

d_o = unadjusted NDT obtained at a pavement temperature, tp, and

k_d = deflection temperature adjustment factor to adjust the pavement to a standard reference temperature of 100°F.

$$k_d = F_d C_d \tag{3.6}$$

where:

F_d = adjustment factor for pavement temperature, and

C_d = a function of the type of existing flexible pavement.

The adjusted deflection values are used to determine the composite modulus of elasticity from:

$$E_c = P/D_p d_{oc} \tag{3.7}$$

Flowchart PCC/AC

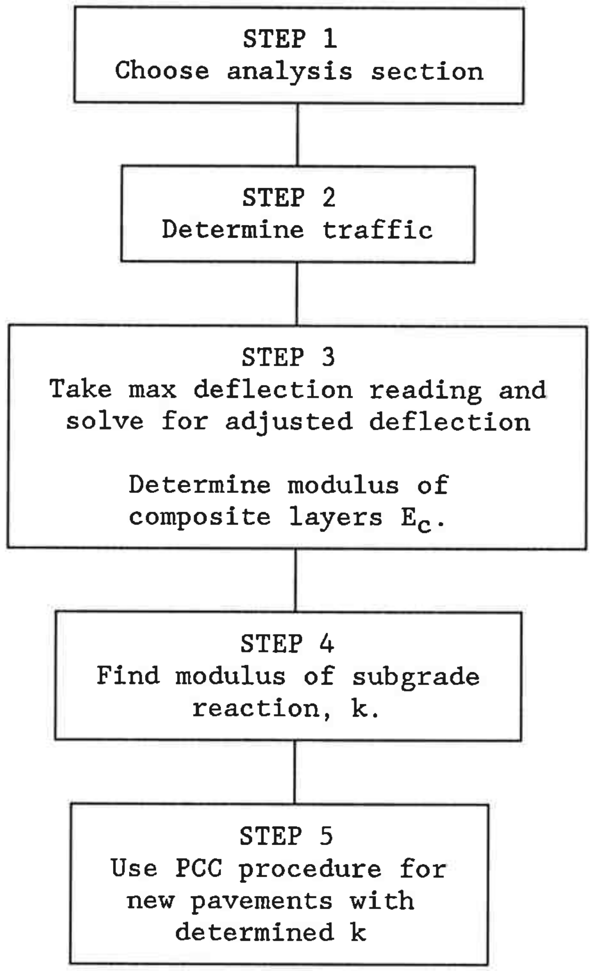


Figure 3.8. Flowchart of Portland Cement Concrete Overlay Over Flexible Pavement.

where:

P = NDT dynamic load (lbs),

D_p = NDT load plate diameter (in.), and

d_{oc} = Adjusted maximum NDT deflection.

The k value is found from E_c and the design proceeds as a new design as outlined in the AASHTO guide Part II.

4.0 FIELD EVALUATION - 1986

This chapter describes the field evaluation conducted as a part of this study during the summer of 1986. Three projects were selected for evaluation to verify the AASHTO relationships. The first section describes the projects and the second section presents the results from field testing and overlay design calculations. Comparisons are made between the 1986 AASHTO design and more traditional procedures. The third section presents the data collected on joints in portland cement concrete pavements. Finally, potential problems of the AASHTO procedure are addressed.

4.1 Project Descriptions

Three sites were selected for the evaluation phase: two asphalt concrete pavements (one relatively new pavement with a cement-treated base, and an older pavement with an untreated base), and one portland cement concrete pavement. These three sites were chosen on the basis of accessibility and available information for design purposes. Figure 4.1 shows the location of the sites while Table 4.1 summarizes the pavement cross sections and pavement conditions for each project. For each project, a test section of 1000 ft (304.8 m) was chosen for deflection testing. Deflection values were taken for every 50 ft (15.24 m) of the test section. All three deflection devices, FWD, Dynaflect, and Benkelman Beam, were employed. Condition rating factors (C_X) and PSR values were assigned to each pavement section to rate the structural adequacy and ride quality. Discussion of these ratings are in Chapter 3.

Three cores were taken on each project (at the beginning, middle, and end of each section) for determining the modulus of the surface layer. Modulus

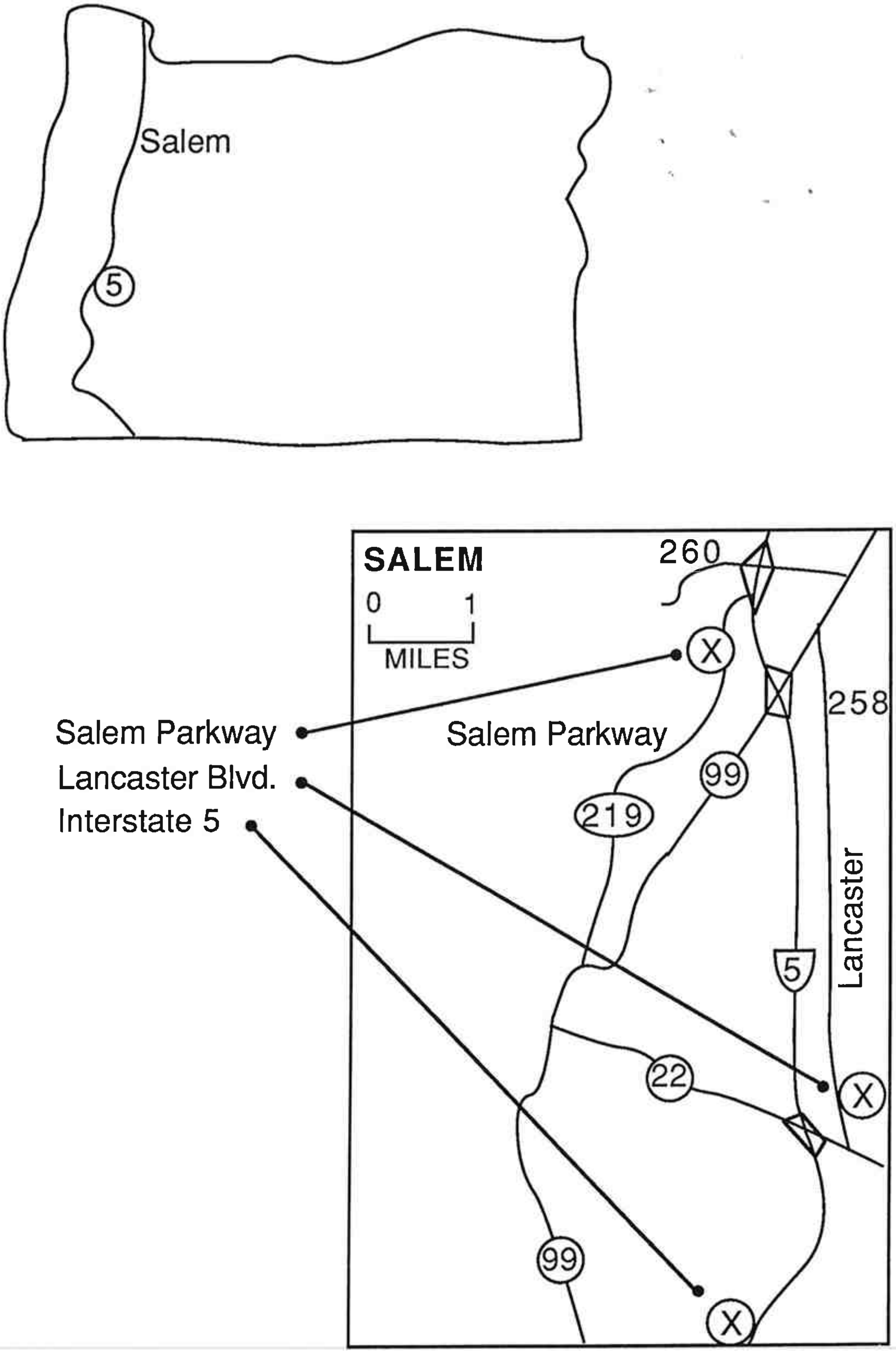


Figure 4.1. Location Map of Project Sites.

Table 4.1. Summary Table of Pavement Conditions at Time of Testing.

Project	Cx	PSR	Cumulative Traffic ($x10^6$ ESAL's)	Ages (years)	Cross Section (inches)	Laboratory Modulus Values (ksi)	Subgrade R-Value
Salem Parkway	1.0	4.5	.42	3	3.5 AC 10 CTB 6 CTS	863 2,280	9
Lancaster Blvd.	4.0	2.1	.63	15	3.5 AC 18 Base	1,652	6
Interstate 5	3.3	2.8	18.5	27	8 PCC 12 Base	2,869	13

Note: 1) Cx is a condition rating in which values vary from 1 (excellent) to 5 (very poor). See Chapter 3 for description.

2) PSR is the AASHTO Pavement Serviceability Rating. See Chapter 3 for description.

(1 ksi = 6.895 N/m^2)

values for the asphalt concrete and cement-treated base cores were determined from 4-in. cores using diametral resilient modulus testing apparatus (ASTM D-4123). Modulus values for the portland cement concrete were obtained from compression tests using 4-in. cores. Additional explanation of the test procedures is in Appendix B. Table 4.1 summarizes the results of modulus tests.

Details of all test data are given in Appendix B; only summaries are contained in this chapter.

4.2 Determination of Overlay Requirements

This section outlines the necessary steps taken to determine the overlay requirements for each project using the flowcharts presented in Chapter 3. Detailed steps are given in Appendix C. Worked examples are presented in Appendix D.

4.2.1 SN_{xeff} Determination

AASHTO presents two alternatives for determining SN_{xeff} of the pavement (NDT Method 1 and NDT Method 2). NDT Method 1 uses the deflection basin produced from a series of sensors from either the FWD or Dynaflect. The modulus is determined for each layer on the assumption that there exists one unique set of modulus values which will predict a particular deflection basin. These modulus values are determined from backcalculation programs and then correlated with layer coefficient values. Determination of the modulus values from BISDEF is influenced by the chosen depth of the rigid layer below the subgrade, as can be seen in Table 4.2. Modulus values were determined for the cases in which the laboratory surface modulus values were fixed in the BISDEF program. The layer thickness multiplied by the layer coefficient for each

Table 4.2. Effect of Depth of Rigid Layer on Layer Modulus Using BISDEF.

Project	Thickness to the Rigid Layer (in.)	Modulus (psi)		
		Surface*	Subbase	Subgrade
Salem	72	863,000	385,753	4,974
	240		245,016	17,290
Lancaster	72	1,652,000	6,211	9,255
	240		5,277	20,917
Interstate 5	72	2,869,000	277,510	3,359
	240		157,978	16,937

(in. = 2.54 cm; 1 psi = 6,895 N/m^2)

*Determined from laboratory tests.

pavement layer are summed to obtain the SN_{xeff} of the pavement.

SN_{xeff} (for NDT method 1) is calculated using both the FWD and Dynaflect data as shown in Tables 4.3 through 4.5 for selected deflection locations. These locations are grouped by similar maximum deflection values (i.e., the average, relatively high, or relatively low). The equivalent diameter of the loading plate for the Dynaflect was determined from a footprint of the loaded area of the Dynaflect wheels and a conversion of it to a circular area (Appendix B).

NDT Method 2 uses the maximum deflection value and a regression equation to calculate SN_{xeff} (Appendix C). SN_{xeff} values for NDT Method 2 using the FWD data are given in Tables 4.6 through 4.8. The Dynaflect data were not used for this analysis.

Both NDT Methods 1 and 2 were evaluated to determine the most accurate and time efficient method of calculating SN_{xeff}. Tables 4.9 through 4.11 demonstrate the change in SN_{xeff} for the various procedures.

4.2.2 Calculation of SN_y and F_{RL}

Once SN_{xeff} is established for the existing pavement section, SN_y and F_{RL} are determined as described in Chapter 2 and Appendix C. SN_y is determined to accommodate future expected traffic and serviceability levels. The AASHTO guide provides the designer with a choice of reliability levels. The reliability chosen can affect the recommended SN_y value. SN_y also varies for a given subgrade modulus and future traffic expectations as can be seen in Table 4.12.

Finally, F_{RL} must be determined from one of the several methods outlined in Chapter 2. The effect of three different approaches on F_{RL} and final overlay design thickness, is shown in Table 4.13. For purposes of this study,

Table 4.3. SN_{xeff} Comparisons for Various NDT Equipment. (Salem Parkway)

	NDT #1 FWD BISDEF (With AC Core Samples*)				NDT #1 Dynaflect BISDEF (With AC Core Samples*)			
Deflection Location	E_{CTB} (psi)	E_{CTS} (psi)	E_{SG} (psi)	SN_{xeff}	E_{CTB} (psi)	E_{CTS} (psi)	E_{SG} (psi)	SN_{xeff}
2	679,705	55,260	25,865	3.70	1,247,843	21,882	25,766	4.64
4	1,016,927	21.922	25,282	4.26	826,341	33.961	26,687	3.98
3	571,510	41,276	23,848	3.34	1,264,845	14,177	22,643	4.40
9	440,940	104,833	19,664	3.26	986,813	12,213	19,664	4.20
11	808,916	25,295	17,288	3.92	2,106,327	2,160	13,586	4.40
12	389,791	49,795	18,838	2.70	567,857	16,546	20,195	3.10
14	599,060	41,485	20,820	3.34	1,046,265	13,835	21,349	4.20
10	3,136,014	12,340	24,191	4.40	3,349,232	5,204	21,977	4.34

(1 psi = 6,895 N/m^2)

*AC Modulus = 863,000 psi

$SN_{xeff} = \Sigma\, a_1h_1 + a_2h_2 + a_3h_3$

e.g.:

Location	SN_{xeff} (for NDT #1, FWD)
8	3.5 (.44) + 10 (.25) + 6 (.03) = 4.22
10	3.5 (.44) + 10 (.28) + 6 (.01) = 4.40

Table 4.4. SN_{xeff} Comparisons for Various NDT Equipment. (Lancaster Blvd.)

Deflection Location	NDT #1 FWD BISDEF (With Core Samples) E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	NDT #1 Dynaflect BISDEF (With Core Samples) E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}
2	7,856	8,750	2.26	15,008	8,607	2.80
3	7,645	9,743	2.26	13,712	9,338	2.62
16	6,538	9,116	2.26	10,466	11,541	2.35
17	9,317	6,810	2.26	18,505	7,972	3.23
5	32,841	8,631	4.06	27,479	8,362	3.84
6	32,300	7,843	4.06	27,583	7,138	3.84
14	25,630	7,683	3.70	28,358	7,833	3.99

(1 psi = 6,895 N/m^2)

$SN_{xeff} = \Sigma\ a_1h_1 + a_2h_2 + a_3h_3$

e.g.:

Location	SN_{xeff}
2	3.5 (.44) + 18 (.04) = 2.26
5	3.5 (.44) + 18 (.14) = 4.06

Table 4.5. SN_{xeff} Comparisons for Various NDT Equipment. (Interstate 5)

Deflection Location	NDT #1 FWD BISDEF (With PCC Core Samples*) E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	NDT #1 Dynaflect BISDEF (With PCC Core Samples*) E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}
1	58,957	25,574	7.68	12,939	16,843	6.00
11	453,544	10,042	7.68	41,504	16,359	7.44
12	172,084	16,076	7.68	73,844	12,388	7.68
14	73,792	9,689	7.68	8,837	18,998	5.64
10	39,199	19,716	7.32	135,873	9,184	7.68
15	136,217	22,656	7.68	39,359	20,305	7.32

Note: $SN_{xeff} = SN_{xeff\text{-}rp} + .8D_{xeff}$

*PCC modulus = 2,869,000 psi

(1 psi = 6,895 N/m^2)

Table 4.6. SN_{xeff} Using NDT Method 2 (Salem Parkway).

Deflection Location	Height (in.)	Load (lbs)	Maximum FWD Deflection ($x10^{-3}$ in.) (Measured)	Subgrade Modulus (psi)	SN_{xeff}
2	19.5	9,000	5.18	20,833	7.46
4	19.5	9,000	5.13	20,677	7.53
3	19.5	9,000	5.72	19,504	7.13
9	19.5	9,000	6.46	16,082	7.03
11	19.5	9,000	6.64	14,139	7.24
12	19.5	9,000	7.21	15,406	6.61
14	19.5	9,000	6.22	17,028	7.07
10	19.5	9,000	4.18	19,784	8.88

(1 in. = 2.54 cm; 1 psi = 6,895 N/m^2)

Table 4.7. SN_{xeff} Using NDT Method 2 (Lancaster Blvd.).

Deflection Location	Height (in.)	Load (lbs)	Maximum FWD Deflection ($x10^{-3}$ in.) (Measured)	Subgrade Modulus (psi)	SN_{xeff}
2	21.5	9,000	28.40	13,740	3.31
3	21.5	9,000	28.02	14,915	3.29
16	21.5	9,000	28.49	13,822	3.30
17	21.5	9,000	28.16	11,575	3.42
5	21.5	9,000	15.22	17,990	4.33
6	21.5	9,000	15.92	16,699	4.29
14	21.5	9,000	18.07	15,546	4.07

(1 in. = 2.54 cm; 1 psi = 6,895 N/m^2)

Table 4.8. SN_{xeff} Using NDT Method 2 (Interstate 5, PCC).

Deflection Location	Height (in.)	Load (lbs)	Maximum FWD Deflection ($x10^{-3}$ in.) (Measured)	Subgrade Modulus (psi)	SN_{xeff}
1	20.0	9,000	4.92	21,182	7.78
11	20.0	9,000	5.03	17,067	8.31
12	20.0	9,000	5.02	18,360	8.09
14	20.0	9,000	4.93	19,932	7.94
10	20.0	9,000	5.89	17,118	7.42
13	20.0	9,000	9.59	20,350	5.15

(1 in. = 2.54 cm; 1 psi = 6,895 N/m^2)

Table 4.9. SN_{xeff} Determination Using FWD (Salem Parkway).

Deflection Location	BISDEF (With AC Core Samples) E_{AC} (psi)	E_{CTB} (psi)	E_{CTS} (psi)	E_{SG} (psi)	SN_{xeff}	BISDEF (Without Core Samples) E_{AC} (psi)	E_{CTB} (psi)	E_{CTS} (psi)	E_{SG} (psi)	SN_{xeff}
2	863,000	679,705	55,260	25,865	3.70	348,469	931,649	75,671	25,865	4.17
4		1,016,927	21,922	25,282	4.26	913,111	843,734	27,675	25,282	3.92
3		571,510	41,276	23,848	3.34	2,989,338	243,487	73,950	23,848	2.44
9		440,940	104,833	19,664	3.26	364,805	496,724	201,921	19,664	4.00
11		808,916	25,295	17,288	3.92	836,790	2,109,737	206,570	17,288	5.60
12		389,791	49,795	18,838	2.70	435,024	685,396	23,071	18,838	3.43
14		599,060	41,485	20,820	3.34	1,651,514	430,532	47,671	20,820	2.68
10		3,136,014	12,340	24,191	4.40	387,573	5,716,788	5,424	24,191	4.24

(1 in. = 2.54 cm; 1 psi = 6,895 N/m^2)

Table 4.10. SN_{xeff} Determination Using FWD.
(Lancaster Blvd.)

	NDT #1						NDT #2	
	BISDEF (With AC Core Samples)			BISDEF (Without Core Samples)				
Deflection Location	E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	E_{SG} (psi)	SN_{xeff}
2	7,856	8,750	2.26	15,600	7,200	2.76	13,740	3.31
3	7,645	9,743	2.26	15,100	7,900	2.76	14,915	3.29
16	6,938	9,116	2.26	13,000	7,400	2.53	13,822	3.30
17	9,317	6,810	2.26	16,300	6,100	2.89	11,575	3.42
5	32,841	8,631	4.06	46,300	8,200	4.87	17,990	4.33
6	32,300	7,843	4.06	45,800	7,400	4.87	16,699	4.29
14	25,630	7,683	3.70	35,400	7,500	4.42	15,546	4.07

(1 psi = 6,895 N/m^2)

Table 4.11. SN_{xeff} Determination Using FWD.
(Interstate 5, PCC)

	NDT #1							NDT #2	
	BISDEF (With PCC Core Samples)			BISDEF (Without Core Samples)					
Deflection Location	E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	E_{AC} (psi)	E_{BASE} (psi)	E_{SG} (psi)	SN_{xeff}	E_{SG} (psi)	SN_{xeff}
1	58,957	25,574	7.68	2,608,844	48,489	25,644	7.52	21,182	7.78
11	453,544	10,042	7.68	4,329,202	111,444	18,653	8.48	17,067	8.31
12	172,084	16,076	7.68	4,505,473	26,984	21,507	8.48	18,360	8.09
14	73,792	9,689	7.68	3,996,153	24,786	24,286	7.28	19,932	7.94
10	39,199	19,716	7.32	2,666,944	56,947	18,587	7.52	17,118	7.42
15	136,217	22,656	7.68	4,185,419	31,046	24,805	7.68	20,350	5.15

Note: SN = $SN_{xeff-rp}$ + .8 D_{xeff} for backcalculation.

(1 psi = 6,895 N/m^2)

Table 4.12. Sensitivity of the Future Structural Capacity (SNy) to Reliability.

	Subgrade Modulus (psi)			
	5,000 Traffic ESAL's		20,000 Traffic ESAL's	
Reliability %	$1x10^6$	$5x10^6$	$1x10^6$	$5x10^6$
95	4.0	5.2	2.4	3.1
90	3.9	4.9	2.3	2.9
85	3.8	4.8	2.3	2.8

Note: Using S_o = .35 and Δpsi = 1.7

(1 psi = 6895 N/m^2)

Table 4.13. Effect of Remaining Life (F_{RL}) on Overlay Thickness.

Approach	R_{LX}	F_{RL}	AC Overlay (in.)
R_{LY} = .20			
NDT	.74	.80	4.78
Traffic	.49	.65	6.01
Serviceability	.30	.58	6.59
R_{LY} = .40			
NDT	.74	.87	4.21
Traffic	.49	.74	5.27
Serviceability	.30	.65	6.01
R_{LY} = .60			
NDT	.74	.94	3.54
Traffic	.49	.84	4.45
Serviceability	.30	.77	5.03
R_{LY} = .80			
NDT	.74	.99	3.22
Traffic	.49	.94	3.63
Serviceability	.30	.90	3.96
F_{RL} = 1.0			3.14

Note: Values determined assuming SN = 5.0, SN_{xeff} = 3.62 as from Example N-10 in the 1986 AASHTO Guide.
(1 in. = 2.54 cm)

the NDT approach was always used. A reliability factor of 95% and standard density of 0.35 was also used.

4.2.3 Overlay Requirements

Knowing the SN_y, SN_{xeff}, and F_{RL}, these values are directly substituted into the general overlay equation:

$$SN_{oL} = SN_y - F_{RL} (SN_{xeff})$$

Tables 4.14 and 4.15 summarize the recommended flexible overlay design thicknesses for the AASHTO procedure as well as for other traditional procedures discussed in Appendix A. Worked examples are contained in Appendices D and E.

Determination of PCC slab thickness for all three projects is shown in Table 4.16. These procedures involved assigning a k value to the existing structure and treating it as a new design. Flowcharts are in Chapter 3.

4.3 Void Detection and Joint Efficiency for Portland Cement Concrete

For portland cement concrete pavements, joint efficiency and void detection under slabs at the joints are of major concern. If the maximum deflection obtained directly under the load on the leave slab is divided by the maximum deflection obtained on the approach slab, the ratio is a measure of the presence of a void (Figure 4.2). The deflection is generally greater under the leave slab. If the ratio is significantly higher (over 2-3) (38), it is possible pumping has occurred at the joint which has resulted in a void.

Joint efficiency is evaluated in terms of the ratio of the maximum deflection directly beneath the load divided by the deflection of the second sensor (one foot away) on the other slab. Under normal operating circumstances with the Dynaflect, the second sensor precedes the first sensor located on

Table 4.14. FWD Thickness Comparisons (Inches) for Various Overlay Procedures (Salem Parkway).

Procedure	Traffic ($x10^6$ ESAL's) 1	3	7	10	15
AASHTO NDT Method 1 (with cores)	-	0	0	0	0
CALTRANS	-	0	0	0	0
Asphalt Inst.	-	0	0	0	0
FHWA-ARE	-	0	0	0	0
ODOT	-	0	0	0	0

(1 in. = 2.54 cm)

Table 4.15. FWD Thickness Comparisons (Inches) for Various Overlay Procedures (Lancaster Blvd.).

Procedure	Traffic ($x10^6$ ESAL's) 1	3	7	10	15
AASHTO NDT Method 2	1.5	2.6	4.2	4.4	5.1
CALTRANS	2.0	2.5	4.0	4.5	5.0
Asphalt Inst.	0.0	1.5	3.0	3.5	4.0
FHWA-ARE	1.0	2.5	3.0	3.5	4.2
ODOT	2.6	3.8	4.6	4.9	5.0

(1 in. = 2.54 cm)

Table 4.16. Portland Cement Concrete Overlay Requirements for the Selected Projects.

Project	Traffic (ESAL's $x10^6$)	AASHTO 1986 PCC Design Thickness (in.)	AASHTO 1981 PCC Design Thickness (in.)
Salem	3.2	7.5	7
Lancaster	1.0	7	6
Interstate 5	79.0	8	7

(1 in. = 2.54 cm)

the approach slab (Figure 4.3). Since the FWD has the first sensor followed by the second sensor on the approach slab (Figure 4.4) it was decided to turn both pieces of equipment around and test in both directions to obtain comparable results. Ratios with values larger than 2 generally are considered to indicate joints with poor load transfer capabilities (2). Tables 4.17 through 4.19 demonstrate the variation that may exist depending upon the equipment used and the direction in which the test is performed. Discussion of these results are in section 4.4.

4.4 Discussion of Results

Table 4.2 demonstrates the effect of the placement of the rigid layer in the BISDEF program. The Army Corps of Engineers recommends placement of this layer at 20 ft below the subgrade.

The effective structural number (SN_{xeff}) determination using NDT method 1 was difficult to obtain for the particular projects. For Salem Parkway, modulus values determined by BISDEF revealed the cement-treated base (CTB) was much stiffer than the cement-treated subbase (CTS). For Interstate 5, modulus values for the untreated base varied from 40,000 to 450,000 psi (275,800 to 3,102,750 KN/m^2). In the future, better estimates of base and subbase modulus values could be accomplished by fixing subgrade layer modulus (from regression equation such as Ullidtz) as well as the surface layer modulus from cores. Correlations between layer coefficients and moduli provided in the AASHTO guide were often inadequate. Therefore, assumed layer coefficient values were used which made the effective structural number determinations appear to agree. However, this was not the case.

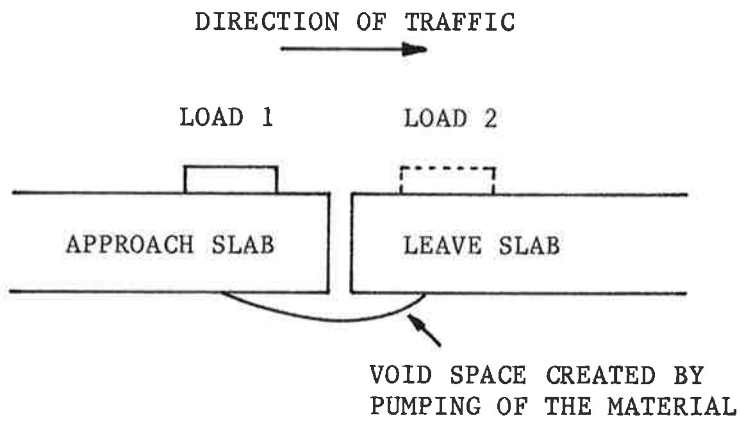


Figure 4.2. Void Detection at Joints in PCC Pavement.

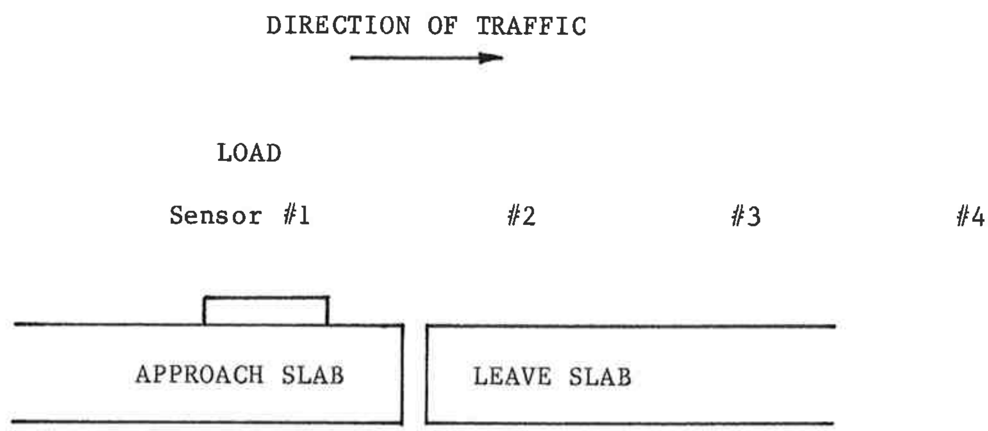


Figure 4.3. Normal Direction Loading for Dynaflect

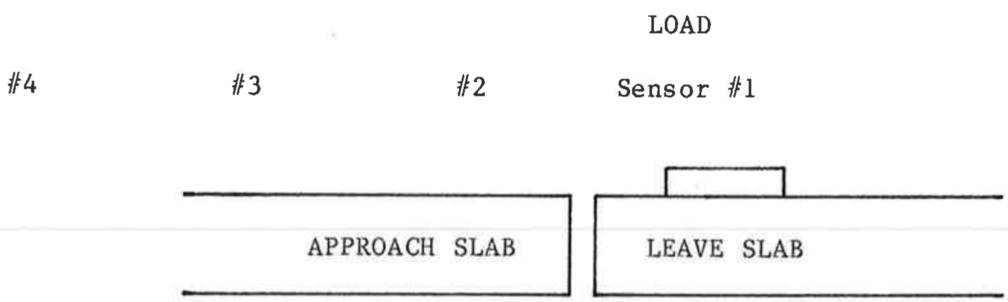


Figure 4.4. Normal Direction Loading for FWD.

Table 4.17. Void Detection at Joints in PCC (Interstate 5).

Joint No.	Deflection (Leave Slab)/Deflection (Approach Slab)	
	FWD (9,000 lbs)	Dynaflect (1,000 lbs)
1	1.69	-
2	1.64	2.42
3	1.38	1.71
4	1.33	1.77
5	1.62	2.29
6	1.04	1.08
7	1.21	1.71
8	1.43	1.44
9	2.51	4.01
10	1.28	2.12
11	1.30	1.74
12	1.29	1.91
13	1.36	2.84
14	1.50	2.79
15	1.32	1.42
16	1.45	1.52
Average	1.46	2.05
	±.33	±.74

(1 lb = 4.448 N)

Table 4.18. Joint Efficiency-Interstate 5 (Load on Leave Slab).

Joint No.	Deflection (Under Load)/Deflection (1 ft away) FWD (9,000 lbs)	Dynaflect (1,000 lbs)
1	2.49	1.39
2	2.64	1.39
3	2.16	1.18
4	2.09	1.48
5	2.29	1.37
6	1.50	1.27
7	1.63	1.36
8	2.01	1.32
9	3.64	3.79
10	1.75	1.27
11	1.72	1.17
12	1.82	1.66
13	2.25	1.99
14	2.29	1.44
15	1.92	1.31
16	2.08	1.47
Average	2.14	1.55
	±.51	±.63

(1 lb = 4.448 N)

Table 4.19. Joint Efficiency-Interstate 5 (Load on Approach Slab).

Joint No.	Equipment FWD (9,000 lbs)	Equipment Dynaflect (1,000 lbs)
1	1.30	1.77
2	1.43	1.57
3	1.16	1.56
4	1.64	1.52
5	1.51	1.38
6	1.37	1.26
7	1.51	1.43
8	1.11	1.35
9	1.70	1.69
10	1.22	1.36
11	1.25	1.24
12	1.50	1.49
13	1.36	1.37
14	1.48	1.29
15	1.32	1.28
16	1.23	1.22
Average	1.38	1.42
	±.17	±.16

(1 lb = 4.448 N)

Drastic differences in moduli values were found for various sets of deflections (Appendix B). For example, the modulus of the subgrade and other layers were significantly different throughout the test section.

SN_{xeff} was also determined using regression equations for NDT Method 2. These equations can be found in Appendix C. The basis for these equations involves solving for the subgrade modulus with the use of Ullidtz equations (44). This resulted in a high effective structural number for the thicker pavement sections (Salem Parkway and Interstate 5) and the values were fairly reasonable for the lower volume, thinner pavement section, Lancaster Blvd. The Ullidtz equations are based on the fact that the equivalent thickness of the pavement structure must be less than the distance to the furthest sensor location, 36 in. (91.44 cm). This was the case for Lancaster Blvd. but not for Salem Parkway and Interstate 5. The equivalent thicknesses for each of these pavements were calculated to be 77.9 in. (197.9 cm) and 53.7 in. (136.4 cm), respectively. Therefore, reasonable confidence can be placed in the effective structural number determination for Lancaster Blvd., but not for Salem Parkway or Interstate 5. The deflection sensors would have to be extended to the minimum required distance, 6 ft (1.83 m), or an alternate procedure should be used. Another assumption built into the development of NDT Method 2 is that the base material has a modulus of 30,000 psi (206,800 Kn/m^2). So if NDT Method 2 is to be used, adjustments need to be made to the equations for treated base materials. It is noted, though, that the calculated subgrade modulus values were much more consistent over the entire project length for this procedure in all cases.

Unlike other overlay design procedures, the reliability level can be chosen depending upon the structure and the anticipated traffic. The importance of choosing a particular reliability level was shown in Table 4.12. The SN_y (and, therefore, the thickness) varied more for lower subgrade modulus with high traffic loads than for any other condition. As expected, the difference between the 90% to the 95% reliability is much greater than the difference between 85% and 90% reliability.

The choice of other factors also influences the final design thickness, such as the method for determining the remaining life factor. The effect on final pavement thickness is much more significant for pavements which have a low remaining life of the overlay once it has served its expected traffic levels. With the most economical designs, this typically will be the case. From Table 4.13 the most conservative value is the serviceability approach, but this may not always be the case. AASHTO recommends use of the NDT procedure, especially for existing damaged pavements. Calculations for comparison purposes were determined using the NDT approach.

The calculations for the AASHTO procedure are in Appendix D. From these values, one can see that some irregularities showed up in the AASHTO procedure. The original SN_o value was determined using standard layer coefficient values multiplied by the thickness of each layer. This result was lower than the estimated effective structural number at the time of testing. In the case of Salem Parkway, the SN_{xeff} was significantly larger than the SN_o. This affects the R_{LX} factor determined from the remaining life procedure. This shows the remaining life of the existing pavement to be greater than 1.0. Logically this does not make any sense for a pavement which has already been in service for a few years. However, when the values were substituted into the appropriate equation, the design revealed that no overlay is required, which agreed with the Caltrans and Asphalt Institute design procedures. If

SN_{xeff} were smaller than the SN_o, as one might expect, it is likely to show that an overlay is needed since the F_{RL} factor will always be less than 1.0, thus reducing the SN_{xeff}.

Because of these problems, it was decided to employ the use of the mechanistic procedure (FHWA-ARE) to check results. The various recommended overlay design thicknesses for several traffic conditions are shown for each procedure in Tables 4.14 and 4.15. For Salem Parkway, all procedures show no overlay is required, which was to be expected as the pavement was only three years old and showed no sign of distress. The deflection values were smaller and the pavement cross section was substantial. For Lancaster Blvd., AASHTO results closely agree with the Caltrans procedure and were slightly higher than the Asphalt Institute recommendations. Table 4.16 shows the recommended portland cement concrete overlay for each project. These values compare well with 1981 recommendations. They are slightly higher since a reliability level of 95% was chosen. This does demonstrate, though, that the basis for the AASHTO procedure must have some minimum thickness of slab required for all projects since the flexible procedure shows no overlay is required for the Salem Parkway. AASHTO did not give enough credit to the existing flexible pavement. For PCC overlays over PCC pavements, a D_{xeff} of the existing pavement is determined and subtracted from what is required to satisfy traffic requirements. However, for flexible pavements, an appropriate k value was calculated and the overlay was determined from a new pavement design. No other credit was again given to the existing structure. In the case of Salem Parkway, the equivalent thickness of the pavement structure is stronger than Interstate 5 and should have been given at least as much credit. Therefore, for flexible pavements with cement-treated materials, it should be assigned an effective structural capacity similar to an existing rigid pavement.

Since joint efficiency and transfer of loads are important in portland cement concrete pavements, testing was conducted at the joints as well as at the center of the slab. In the past, other agencies have used the Dynaflect to evaluate the efficiency of the joints, but the load was so small that the deflection values seemed inconsistent. It was more difficult to obtain reliable results to determine the ratios for evaluating void detection and joint efficiency. The standard deviations were generally higher for the Dynaflect data. The average value for the ratios with the FWD operating in its normal direction was higher than the Dynaflect value. Both average values for the Dynaflect and the FWD were larger with the load on the leave slab. This was to be expected since generally the leave slab has greater deflections than the approach slab. It is much more difficult with the FWD operating in its normal direction to determine potentially inadequate joints since the load is on the approach slab.

4.5 Limitations of AASHTO

From the limited testing performed, several problems were encountered in the application of the AASHTO 1986 guide. These are listed below:

1. When using NDT Method 1, backcalculating procedure without a fixed surface modulus, the SN_{xeff} of the pavement was not predicted satisfactorily.
2. Layer modulus values determined with backcalculating procedures did not correlate well with given ranges of layer coefficients for specific materials.

3. Backcalculating moduli values for thinner pavement sections, even with fixed surface modulus values, were unsatisfactory. Individual studies would be needed for each pavement structure to determine optimum depth of rigid layer.
4. Inconsistent and unreliable estimates of layer modulus were obtained with BISDEF for thicker as well as thinner pavement sections even with fixed surface moduli.
5. Use of NDT Method 2 for thicker pavement sections requires the location of the furthest sensor to be at least equal to the equivalent thickness of the pavement structure and modifications to the design equations.
6. Remaining life factors determined for a particular section do not agree. It is necessary to use engineering judgment to assign the appropriate factor.
7. Obtaining cumulative traffic for lower volume roads will be more difficult than for Interstates.
8. SN_{xeff} factors are sometimes greater than SN_o resulting in a remaining life of existing pavement greater than 1.0 for the existing pavement.
9. For asphalt overlays over PCC pavements, the SN_y nomograph must be extrapolated for higher volumes of traffic.
10. Not enough consideration is given to potential reflective cracking in both existing and flexible pavements.
11. For portland cement concrete overlays the design slab thicknesses seem excessive compared to the flexible overlay requirements.

12. Not enough credit is given to existing flexible structure for rigid overlays.

13. Many of the figures provided in the AASHTO guide were undocumented as to where they came from and the data which support them.

5.0 RECOMMENDED DESIGN PROCESS

This chapter contains preliminary recommendations for the design of overlays. The AASHTO procedure for rigid pavements is easier to follow and minimum difficulty is encountered in determining the design values. However, preliminary investigation reveals the slab thickness recommendations are high and too much conservatism may be built into the procedure. This procedure should then be carefully reviewed to determine where the excess is built into the design and attempt to eliminate it. In Oregon the primary overlay situation is flexible overlays over flexible pavements so that will be discussed in much greater detail in this chapter. An outline of a design manual is given in Appendix C for each overlay situation and recommendations are only briefly discussed here. Also contained in this chapter are suggestions on implementation of the flexible overlay design procedure in Oregon.

5.1 Recommendations for New Flexible Overlay Design Procedure

Since many problems were encountered in the use of the AASHTO procedure, it is recommended that two different procedures be used to design flexible overlays. One procedure would be NDT Method 2 for thinner pavements (AC over untreated base). NDT Method 1 or the FHWA-ARE mechanistic approach could be employed for thicker pavements (AC over CTB). Flowcharts for the AASHTO procedure are given in Chapter 3. It should be noted AASHTO does not address the problem of reflective cracking for asphalt overlays over existing asphalt concrete pavements.

5.1.1 Lower Volume Roads (Thinner Pavement Sections)

For NDT Method 2, it is recommended that the entire project length be tested instead of choosing a 1000-ft (304.8 m) analysis section. This would eliminate the problem of choosing a uniform stretch of pavement. Instead of testing every 50 ft (15.24 m), the distance between tests could be increased to every 0.1 to 0.2 mile (.17 to .32 km). The Ullidtz equations would then be used to determine the subgrade modulus for pavements with an equivalent thickness of less than 36 in. (91.44 cm). If greater equivalent thickness of pavement is encountered, the sensor should be fully extended to 6 ft (1.83 m). Knowing the subgrade modulus, maximum deflection value, and total thickness of the pavement layers, SN_{xeff} can be determined for each deflection location. All SN_{xeff} values would be averaged to determine the representative SN_{xeff} for each pavement section under design. The SN_o of the in situ pavement can be assessed by assigning standard values to each of the layers. Using the NDT approach for determining remaining life, an R_{LX} factor could be determined from dividing the SN_{xeff} by the SN_o and using the appropriate chart. SN_y is determined from a knowledge of future traffic requirements, subgrade modulus, ΔPSI, reliability level, and standard deviation. For lower volume roads a reliability level of 85% and a standard deviation of .35 would probably be sufficient. The expected traffic for a design period of 20 years would be used along with a chosen ΔPSI to determine the SN_y. The ΔPSI is dependent upon the lowest index which will be tolerated for the pavement under consideration before rehabilitation becomes necessary. For lower volume roads, use $P_t = 2.0$, and a $P_o = 4.2$ for flexible roads. (Note: if a $P_t = 2.0$ and $P_f = 2.0$, then $N_{FY} = y$ and $R_{LY} = 0$.) Once R_{LY} has been calculated from the total number of loads to failure ($P_f = 2.0$), the F_{RL} can be found. All the known values may then be substituted into the appropriate equation and SN_{OL} can be determined. If SN_{OL} is divided by the layer coefficient for asphalt concrete the required overlay thickness can be found.

5.1.2 Higher Volume Roads (Thick Pavement Structures)

It is still possible to use NDT Method 1 for design purposes, but it should be checked against other design procedures until all the relationships are verified. Individual agencies may choose to develop their own layer coefficient correlation charts for materials they are accustomed to using. If this is not desirable, a mechanistic approach such as the FHWA-ARE procedure could be employed. Both procedures are discussed below.

NDT Method 1 - As in the lower volume road procedure, it is suggested that the entire project length be tested instead of choosing an analysis section. Cores should also be taken at the time of testing. Ullidtz equations for determining subgrade modulus could be used to eliminate another unknown for backcalculation procedures to better estimate the modulus of the base and subbase layers. Too much data would be generated if every deflection basin is analyzed, therefore the designer should choose the locations at which the modulus values should be determined. This could be accomplished by using spreadability values and ratios between the sensors to determine statistically different basins. Another approach would be to develop a profile of the project and use those locations which best represent the pavement and check those locations with significantly higher deflections. (Note: Do NOT take an average deflection basin for use in backcalculating procedures. This can lead to erroneous estimates of layer moduli.) Correlation charts are used to determine layer coefficient values. The SN_{xeff} would be calculated from the summation of structural contribution from each layer. In higher volume roads historical cumulative traffic is often available, so both the NDT and traffic remaining life approaches may be utilized to determine R_{LX}. A conservative value of R_{LX} should be chosen to ensure an adequate design thickness. The R_{LY}

and F_{RL} factors are determined from expected future traffic as previously described. The SN_y is determined using $P_t = 2.5$ for higher volume roads and $P_o = 4.5$ for rigid existing pavements. Reliability values of 90 to 95% and standard deviation of .35 are recommended. This will change the ΔPSI value and therefore the required SN_y. These values would all be inserted into the general equation to solve for SN_{OL} and consequently h_{OL}.

Mechanistic Approach - The deflection and the moduli of each layer are determined as suggested above using core data and backcalculating programs. The modulus data then becomes the input for the ELSYM5 computer program. Normal strains in the pavement structure are determined from the known load, modulus values, and layer thicknesses. The tensile strain at the bottom of the asphalt layer and the compressive strain at the top of the subgrade layer are used to determine the fatigue and deformation potential of the pavement, respectively. An overlay modulus is assumed and several trial thicknesses are input into the ELSYM5 program to obtain relationships between normal strains and the thickness of the overlay. From developed relationships between the number of cycles to failure vs. tolerable strain, the maximum tolerable strain level for both fatigue and deformation can be determined for the expected traffic. Entering the tolerable strain values on the appropriate figures (tolerable strain vs. thickness for both fatigue and deformation considerations) will show whether an overlay is required and what thickness it should be.

5.2 Implementation Guidelines

5.2.1 Lower Volume Roads

The NDT Method 2 for lower volume roads can be implemented immediately. The Ullidtz and AASHTO equations should be computerized to obtain the design overlay thickness. The relationships appear sound for determining SN_{xeff} and SN_y. The only problem which may be encountered is the determination of the remaining life factor F_{RL}. ODOT will have to determine which method they feel the most comfortable with to calculate the R_{LX} factor given the available information. The obtained overlay design thickness recommendation could be periodically checked with former procedures. In the future it would be wise to begin estimating SN_o of the original pavement structure to assist in the determination of R_{LX} using the NDT approach.

5.2.2 Higher Volume Roads

It will take much longer to implement the AASHTO procedure for higher volume roads since many relationships given in the 1986 AASHTO guide need to be verified. Prediction of layer moduli needs to be improved and correlations between layer coefficients and modulus values need to be developed or altered. For higher volume roads it would be advisable to implement this procedure in a series of steps.

First, the agency needs to be comfortable with the determined modulus values for each pavement layer. This requires coring and becoming familiar with and confident in resilient modulus testing. ODOT will have to determine how many cores are needed and where they should be taken to determine an appropriate surface modulus for the length of the project. Then ODOT must become familiar with the use of backcalculation programs, both the advantages and limitations. They may want to experiment with several available backcalculating programs. The subgrade modulus value could also be determined from regression equations if NDT sensors are located out far enough and then input as fixed values into backcalculating procedures to eliminate another unknown. Once the materials are characterized, it is recommended that the mechanistic approach be employed initially. This procedure can be utilized since the relationships between strain vs. fatigue and rutting are known to be valid. Periodically, the NDT Method 1 from AASHTO could be used to determine if the given relationships in AASHTO are reasonable. If ODOT realizes there are too many difficulties encountered with implementing NDT Method 1, then the mechanistic approach should be used to design flexible pavements.

If the agency determines that the AASHTO relationships are proving to be valid, then a second implementation step may be taken. By this time the agency should be comfortable with the obtained modulus values, and all that will be required is to assign appropriate layer coefficient values. The agency can use the recommended correlations provided in the AASHTO guide or develop its own. Development of layer coefficient correlation charts will require a great deal of research and laboratory testing.

During this time period, it would also be wise to investigate the F_{RL}. This would require the determination of SN_o, cumulative traffic to date, and the change in the serviceability ratings. All these factors need to be collected to start a data base which would provide for a more reliable estimate of R_{LX}. By the time the agency is ready to implement NDT Method 1, there should be a better understanding of the R_{LX} factor for each pavement. Monitoring could include noting the change in SN, PSI and R_{LX} over time. Relationships could then be developed directly to assist the designer in determining the remaining life factor of the existing pavement.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Preliminary conclusions and recommendations can be made about the 1986 AASHTO procedure for Oregon's use from the data obtained from the three test projects.

6.1 Conclusions

1. The 1986 AASHTO guide has the advantage of choosing reliability levels and assigning remaining life factors to the pavement structure.
2. The layer materials may be characterized for existing pavements, if backcalculation procedures become more reliable.
3. NDT Method 1 could be implemented over time for higher volume roads if a great deal of research is undertaken.
4. NDT Method 2 appears to work well for the lower volume roads and could be employed on higher volume roads with some adjustments made to the equations.
5. The guide allows an appropriate k (modulus of subgrade reaction) value to be assigned to the existing pavement structure for portland cement concrete over asphalt concrete, but it may not give enough credit to existing flexible pavements.
6. PCC overlay thicknesses are generally greater than expected.
7. Minimal consideration of reflective cracking is provided in the AASHTO guide.
8. Several limitations exist for the immediate implementation of this procedure including investigation of AASHTO relationships.

6.2 Recommendations

1. Flexible design procedures for lower volume roads and higher volume roads should be treated separately.
2. Relationships between surface modulus and layer coefficient for NDT Method 1 should be verified with further testing and analysis before implementation.
3. For higher volume roads FHWA-ARE mechanistic approach should be used initially.
4. Monitoring of SN_o, PSI, and R_{LX} should begin for all higher volume highways to establish a confident database.
5. Further testing should include some sensitivity analysis of the importance of the depth of the rigid layer in the BISDEF program especially for thinner pavement sections.
6. Correlation charts for layer coefficient values should be modified or developed individually by a particular agency.
7. Further investigation of PCC slab recommendations is needed before implementation.
8. Further testing should be conducted on the three selected project sites for this report as well as other site locations. This would include expanding the sensors to be certain the furthest sensor is characterizing the subgrade layer and not the pavement structure. Sensors should be increased to 6 ft (1.83 m) especially for pavements with treated bases.
9. Determined subgrade modulus from regression equations should be input into backcalculating procedures as well as surface modulus to better characterize the base and subbase materials.

10. Dynaflect data can be correlated with FWD values and used as input for NDT Method 2 in the future.
11. Implementation of a new overlay design procedure will involve a significant amount of time and research to ensure the desired results are achieved. Caution and engineering judgment during this time period is critical.
12. Oregon should continue to research in other areas for factors which influence overlays (seasonal influences, traffic factors, condition factors and Mayes ridemeter values, and limitation of reflective cracking).

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APPENDIX A

APPROACHES TO OVERLAY DESIGN

This appendix summarizes other overlay design procedures available: deflection-based and mechanistic. Some of these procedures were used as comparisons to the AASHTO procedure. Example calculations are in Appendix E.

A.1 Deflection-Based Methods

There are three basic elements included in deflection-based overlay design procedures. They include: 1) deflection measurements; 2) evaluation of pavement condition; and 3) prediction of future traffic (see Figure A.1). The objective of the deflection testing is to measure the structural properties of the pavement. A known load is applied to the pavement and the deflection response is measured. The response is a function of the thickness of the pavement layers, subgrade strength, environmental conditions, and the loading conditions (e.g. contact pressure total load, and time of loading). The overlay thickness is normally calculated from the maximum deflection under the load, although the use of the shape factor is gradually increasing. Several types of equipment have been used to measure deflection: Benkelman Beam, Dynaflect, Road Rater, and Falling Weight Deflectometer (FWD).

A pavement condition survey is an important part of a deflection-based overlay design procedure. It establishes the need for maintenance and rehabilitation, identifies homogeneous sections of roadway and points out special considerations such as drainage. A homogeneous section refers to a segment of pavement which has nearly the same traffic, age, structural capacity, and performance. Since there are different scales used for

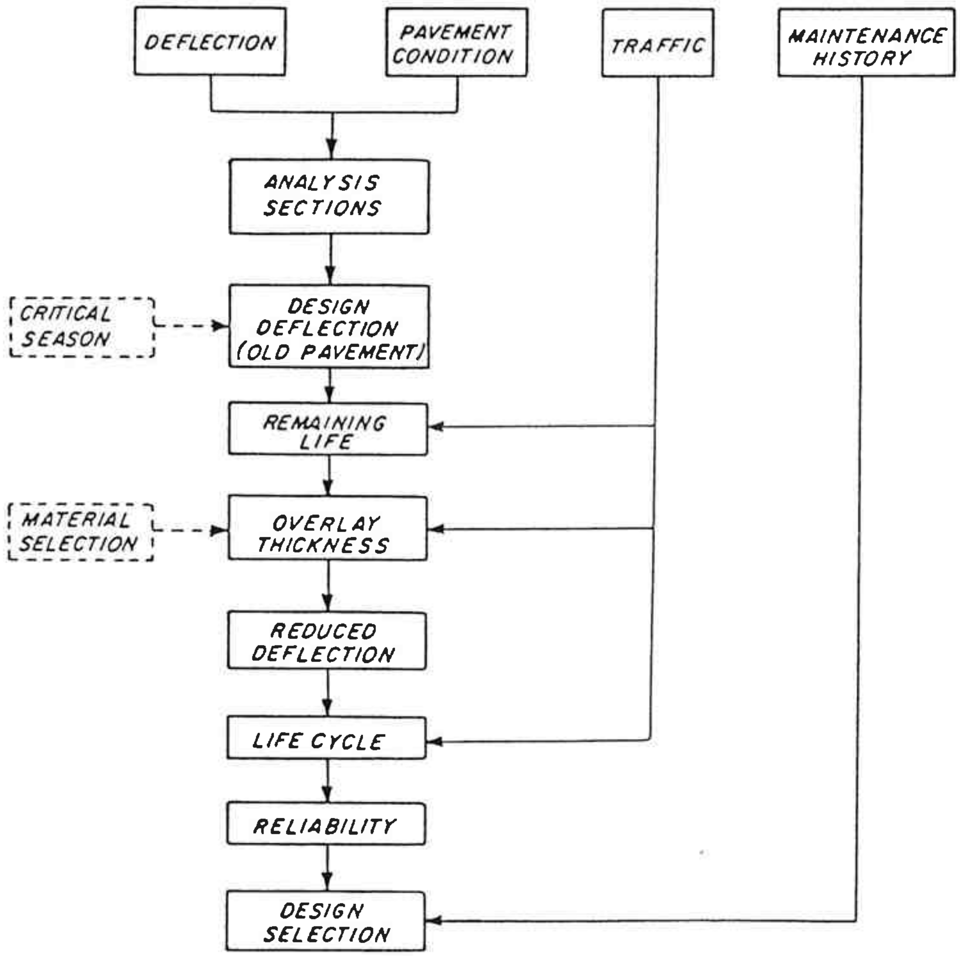


Figure A.1. Methodology Applicable to Overlay Design Using Deflection Measurements (17).

classifying pavement damage, the intervals are a matter of judgment and experience.

Most deflection-based procedures rely on determining two deflections: the design and the tolerable values. The design deflection (d) is a function of the mean (x) and standard deviation (s) of the measured deflection values as follows:

$$d = x + zs \tag{A.1}$$

The z value varies from 2.0 for the Asphalt Institute Method to .84 for Caltrans Method (8). The Asphalt Institute Method also makes adjustments for measurements taken other than the critical time of year and the temperature at which the deflection values are measured are corrected to a temperature of 70°F (21°C). Caltrans believes normalization is not necessary for either temperature or season (9).

Traffic is an important consideration in both overlay design procedures, and is expressed in terms of 18000 lb (80 KN) ESAL. Mixed traffic may be converted into a single design factor by summing all the load combinations and the number of each. The error in estimating overlay thickness or the remaining life may be significant depending upon the reliability of the historical information and future traffic estimates. The tolerable deflection is based on the design ESAL's. The basic philosophy for deflection-based overlay design is to reduce the measured deflection to a tolerable level.

<u>Asphalt Institute Method (6)</u>. Deflection measurements are obtained from the Benkelman Beam or equivalent test equipment. If the Benkelman Beam is not used, correlations between other equipment and the Benkelman Beam can be employed. Many agencies have developed these relationships for the Dynaflect (Table A.1 and Figure A.2). At least ten measurements should be made for a

Table A.1. Conversions for Dynaflect Values to Benkelman Beam.

Agency	Correlation Equation
Forest Service (1975)	$BB = -406.149D^2 + 24.132D + .00017$
Idaho DOT	$BB = 22.5D$
Forest Service (1975)	$BB = -406.149D^2 + 24.132D + .00017$
(1986)	$BB = (76.142889)(D^{1.118363})$
Caltrans	$BB = 18.33D + .004$ (see Figure A.1)
Oregon DOT 1)	$BB = 20X - 4.4$ (D less than 2.5)
2)	$BB = 15(X)^{1.3}$ (D more than 2.5)

BB = beam deflection (in.)

D = dynaflect (in.)

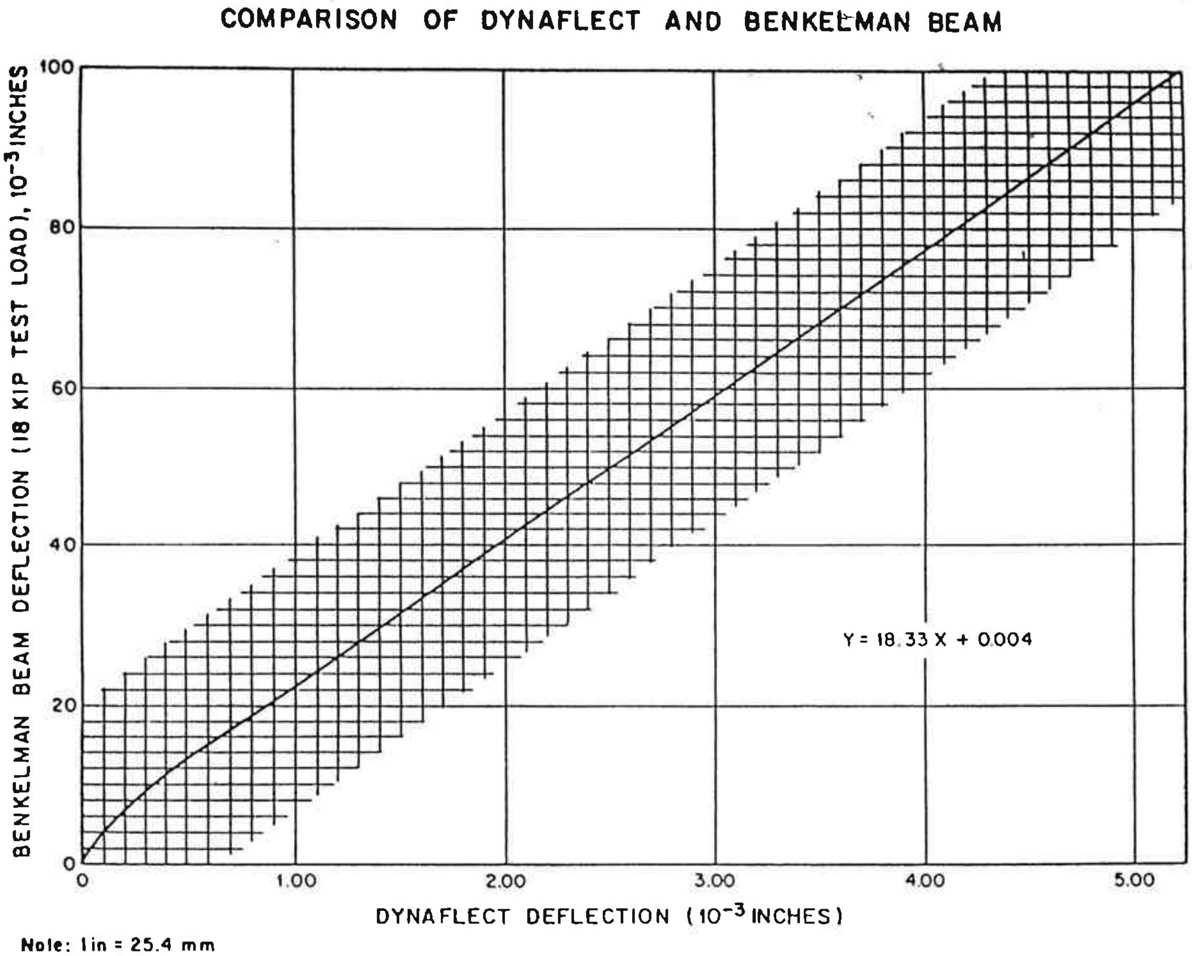


Figure A.2. Dynaflect Readings vs. Benkelman Beam Deflections for Caltrans (9).

particular test section or a minimum of 20 measurements per mile. The deflection measurements are adjusted to a standard temperature of 70°F (21°C).

The design deflection (representative rebound deflection) is calculated from the following equation

$$D_{rrd} = (x + 2s)\ fc \tag{A.2}$$

where:

x = mean deflection,

f = temperature adjustment factor (Figure A.3),

c = critical period adjustment factor, and

s = standard deviation.

The deflection value can be used to estimate remaining life or to determine the required overlay thickness. An estimate of remaining life can be made by entering Figure A.4 with the design deflection. This traffic is termed the permissible traffic. If this value is compared with the amount of traffic which has already been applied to the pavement, the remaining life can be computed.

The anticipated traffic along with the D_{rrd} value are also employed to calculate the required overlay thickness using Figure A.5. The D_{rrd} value is entered on x axis and extended to the anticipated traffic to determine the overlay thickness.

Caltrans Procedure (9). Pavement deflections may be measured using Benkelman Beam, Dynaflect, Road Rater, or Traveling Deflectometers. Homogeneous sections are chosen based on the length on the project. If the project is less than a mile in length, then the entire project is treated as one section.

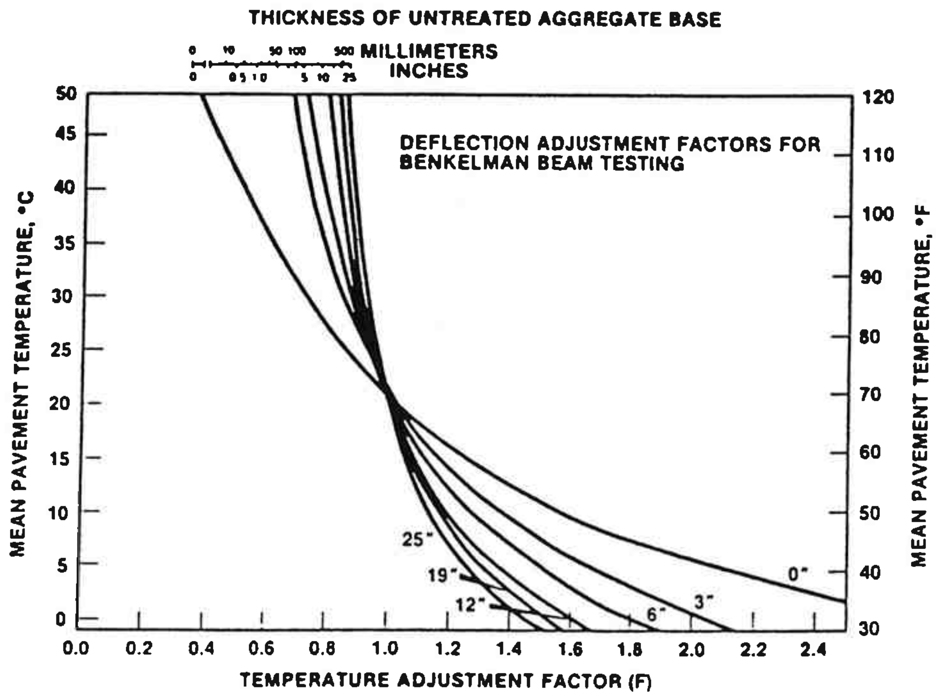


Figure A.3. Temperature Adjustment Factors for Benkelman Beam Deflections (6).

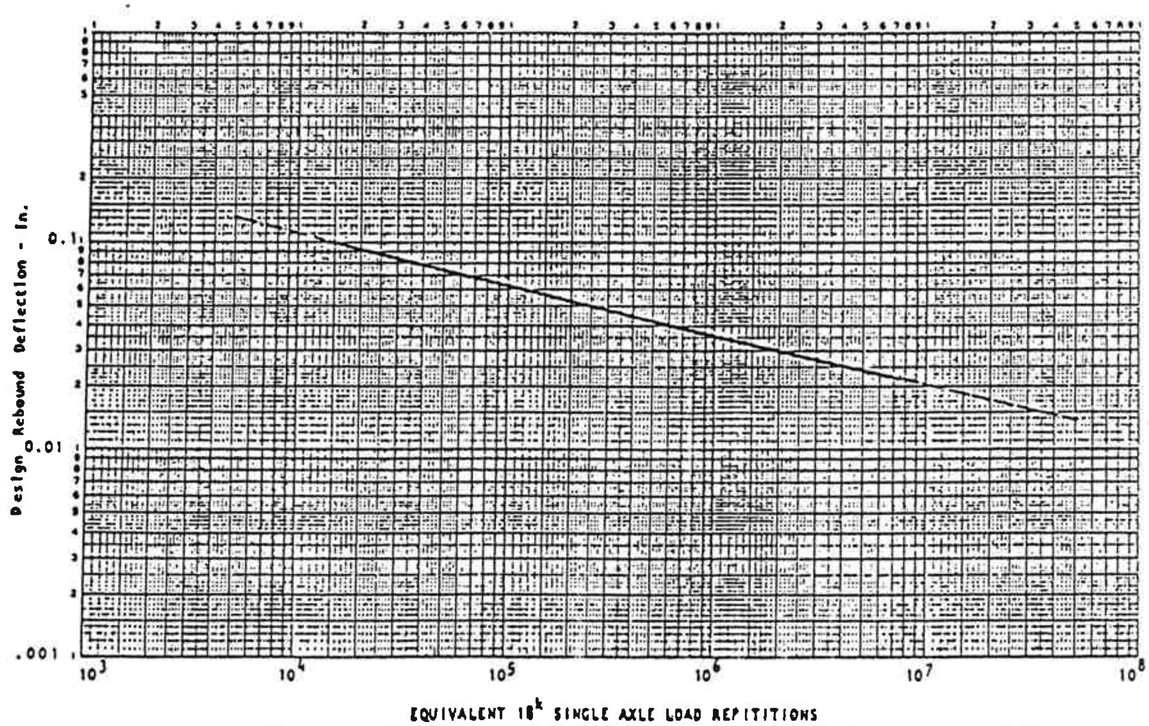


Figure A.4. Design Rebound Deflection Chart (6).

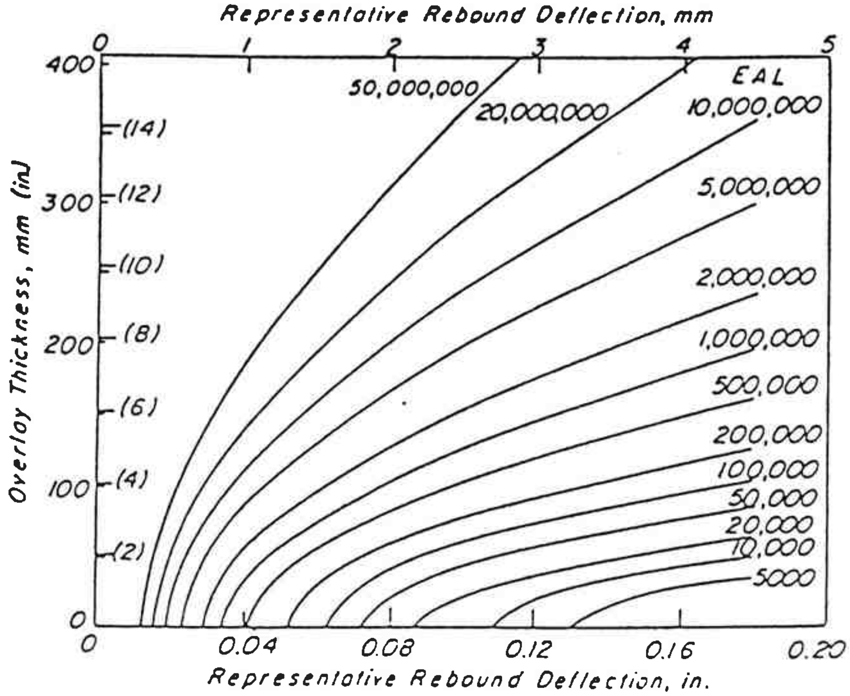


Figure A.5. Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflection from a Measured to a Design Deflection Value (6).

If the project length is greater than one mile, then 1000 ft (304.8 m) sections are selected to represent each mile.

General conditions from a visual survey are documented. Patching, rutting, raveling, and cracking are all noted as to the type and severity. The design deflection is computed from the following equation:

$$D_{80} = x + 0.84s \qquad \text{(A.3)}$$

where:

D_{80} = design deflection value,

x = mean deflection, and

s = standard deviation.

This procedure does not incorporate the remaining life of the pavement into the design. The representative deflection for a particular project length is compared with a tolerable deflection obtained from Figure A.6. If the tolerable deflection is greater than the representative deflection, then an overlay is not required. If the tolerable deflection is less than the representative deflection then the percent reduction in deflection is calculated as follows:

$$\%\ \text{reduction} = \frac{(D_{80} - D_t)}{D_{80}} \times 100 \qquad \text{(A.4)}$$

This value is then entered on Figure A.7 to determine the required gravel equivalency value. The gravel equivalency factor is converted to an equivalent thickness of asphalt concrete by dividing by 1.9. It should be at least half the thickness of the existing asphalt concrete for an untreated base.

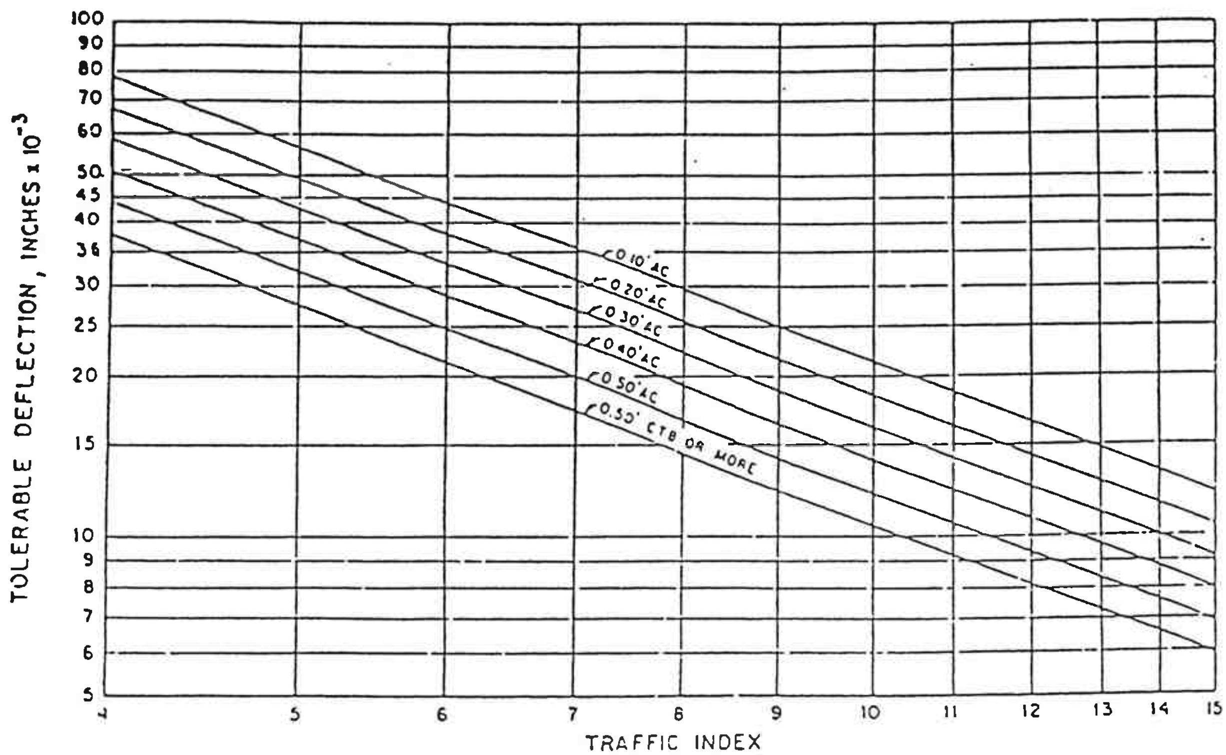


Figure A.6. Tolerable Deflection Chart (9).

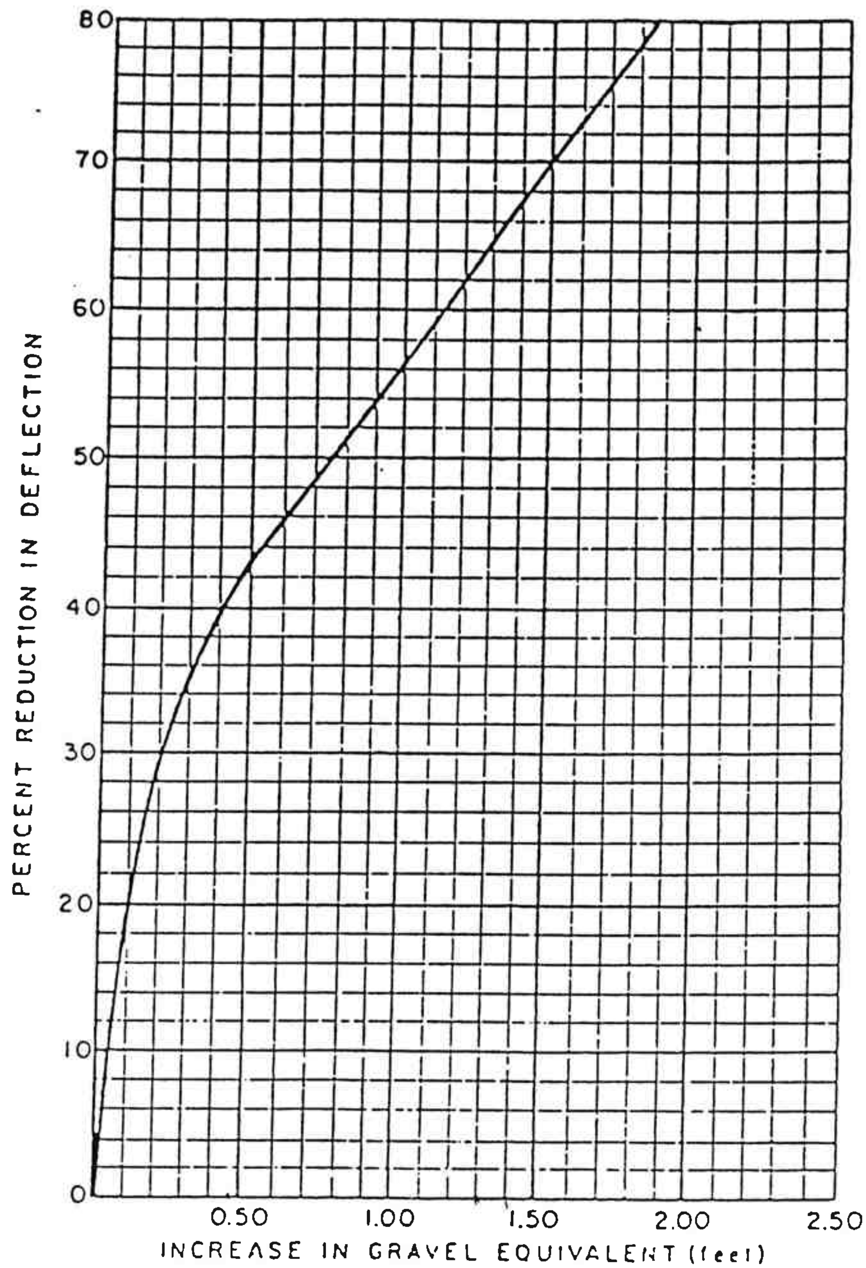


Figure A.7. Reduction in Deflection Resulting from Pavement Overlays (9).

Transport and Road Research Laboratory (TRRL) Procedure (20). In this method, deflections are measured with either the Benkelman Beam or a modified version of the Lacroix deflectograph. An axle load of 14,000 lbs (62.3 KN) is used for both devices. Deflection measurements are taken every 40 to 80 ft (12.2 to 24.4 m) depending upon the condition of the pavement. Measured deflections are adjusted for temperature effects using the chart given in Figure A.8. If deflections are obtained from the deflectograph they are converted to equivalent Benkelman Beam using Figure A.9.

An assessment of remaining life is determined from the representative design deflection, an estimate of traffic and a particular probability of attaining the design life (Figure A.10). The life expectancy is determined from the point which represents the current deflection and total applied traffic and extending it to the probability in achieving the design life to determine the remaining life in standard axles. If the value is in the critical condition, strengthening is required immediately.

Overlay thicknesses are selected from charts such as these given in Figures A.11 and A.12. The expected traffic on the x-axis is projected to the pavement deflection before overlay for an assigned percentile deflection and the overlay thickness is read off the y-axis. A deflection survey of the road demonstrates which parts of the road may need additional thickness for the overlay (Table A.2).

Arizona Overlay Design (45). The Arizona State Department of Transportation developed an overlay design method based on the use of the Dynaflect equipment and empirical, as well as theoretical, concepts (45). Thirty-one variables were considered for their various effects upon the thickness of the overlay for 170 mile post locations. The intent of this analysis was to determine which

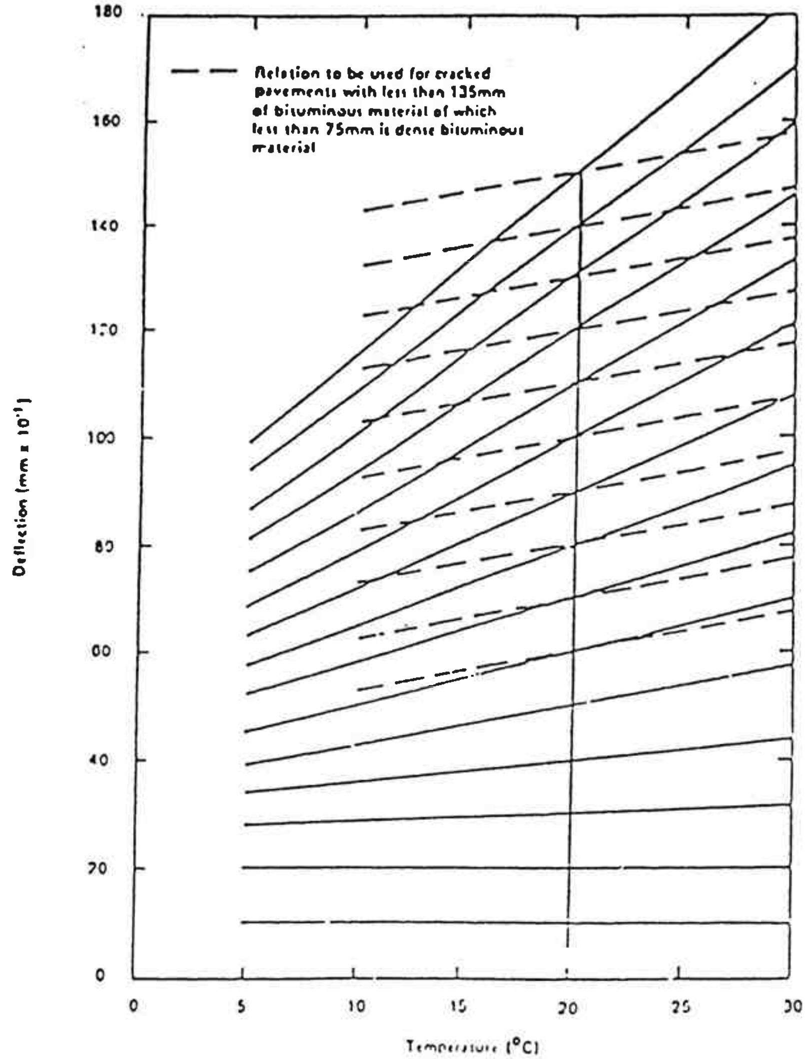


Figure A.8. Relation between Deflection and Temperature for Pavements with less than 135 mm of Bituminous Material, of which less than 75 mm is Dense Bituminous Material (20).

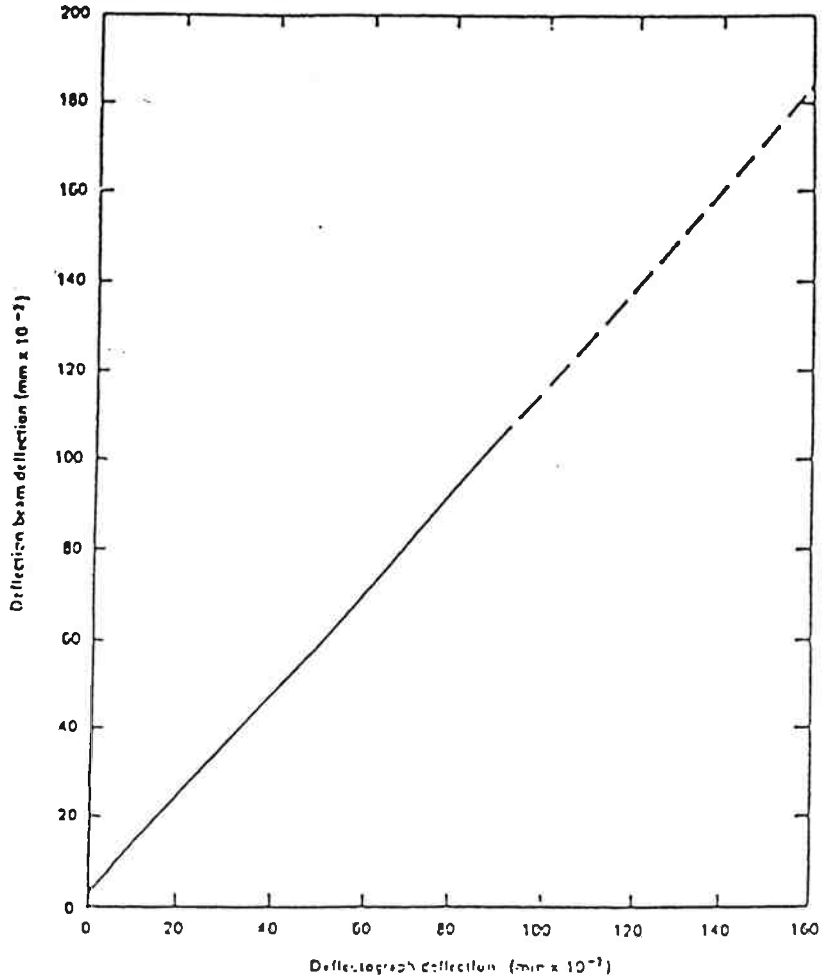


Figure A.9. Correlation Between Deflection Beam and Deflectograph (20).

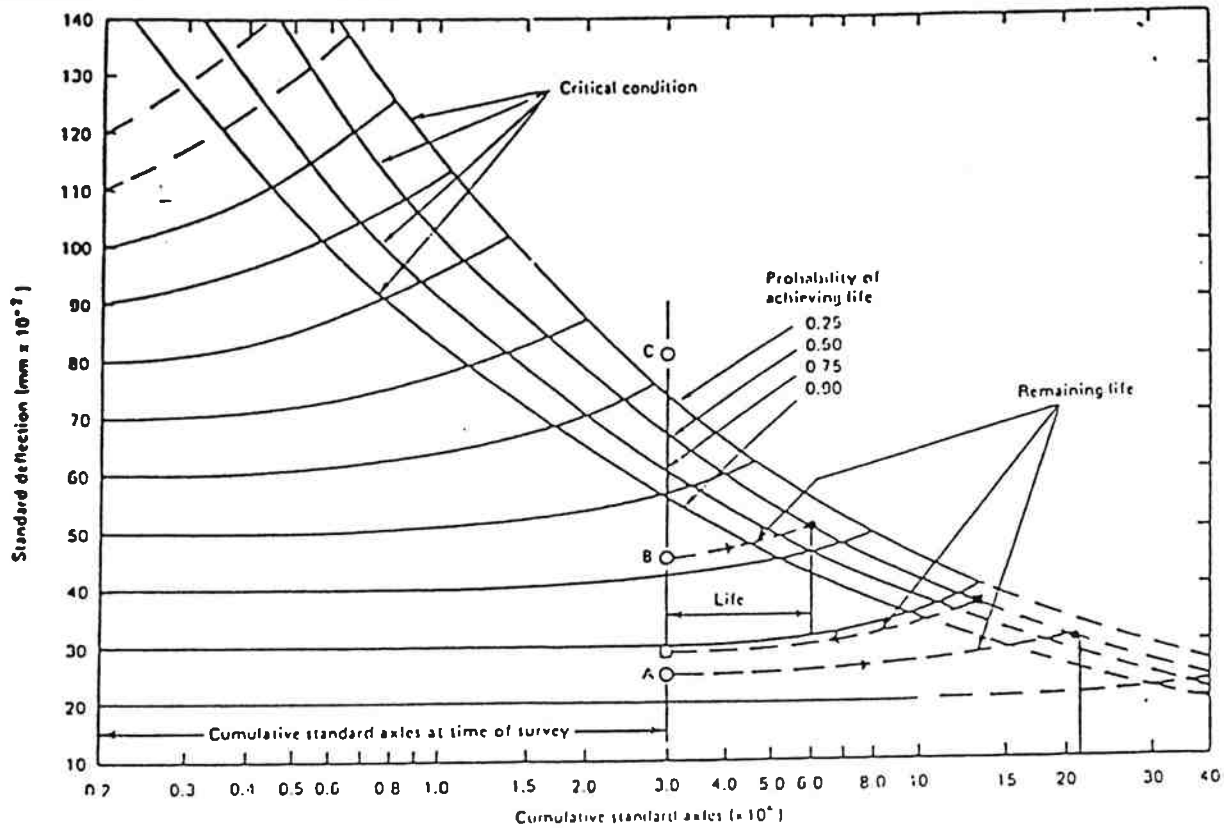


Figure A.10. Relation between Standard Deflection and Life for Pavements with Granular Road Bases Whose Aggregates Exhibit a Natural Cementing Section (20).

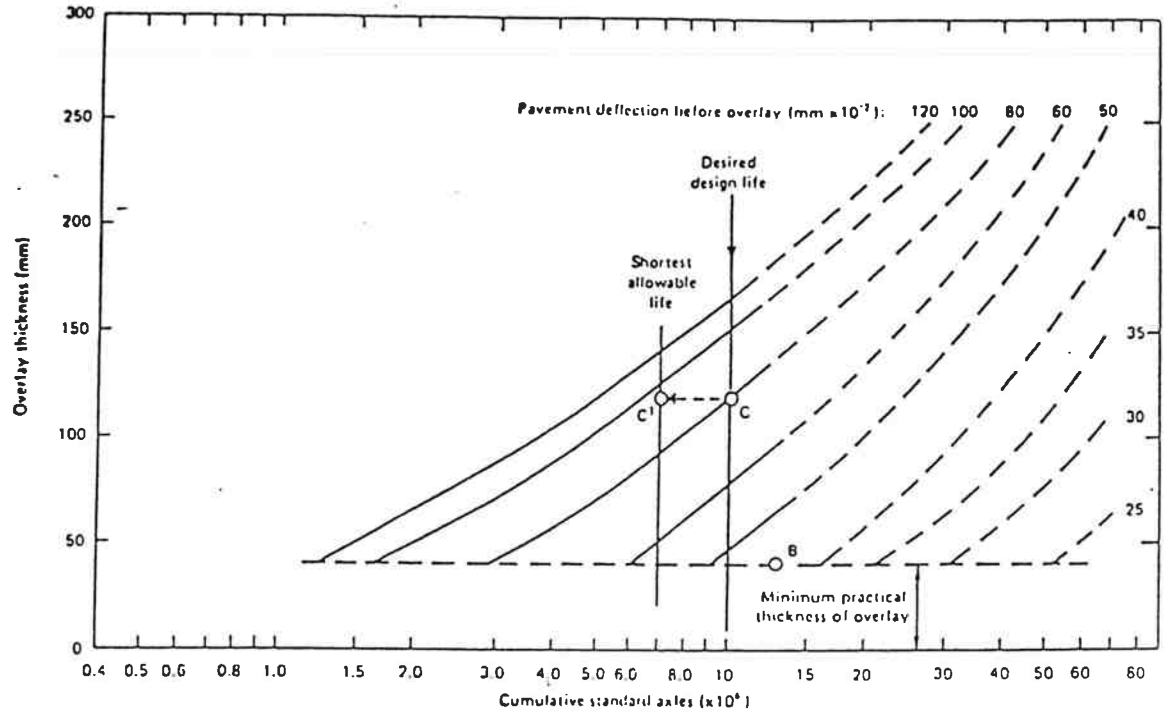


Figure A.11. Overlay Design Chart for Pavements with Granular Road Bases Whose Aggregates have a Natural Cementing Action (.50 Probability) (20).

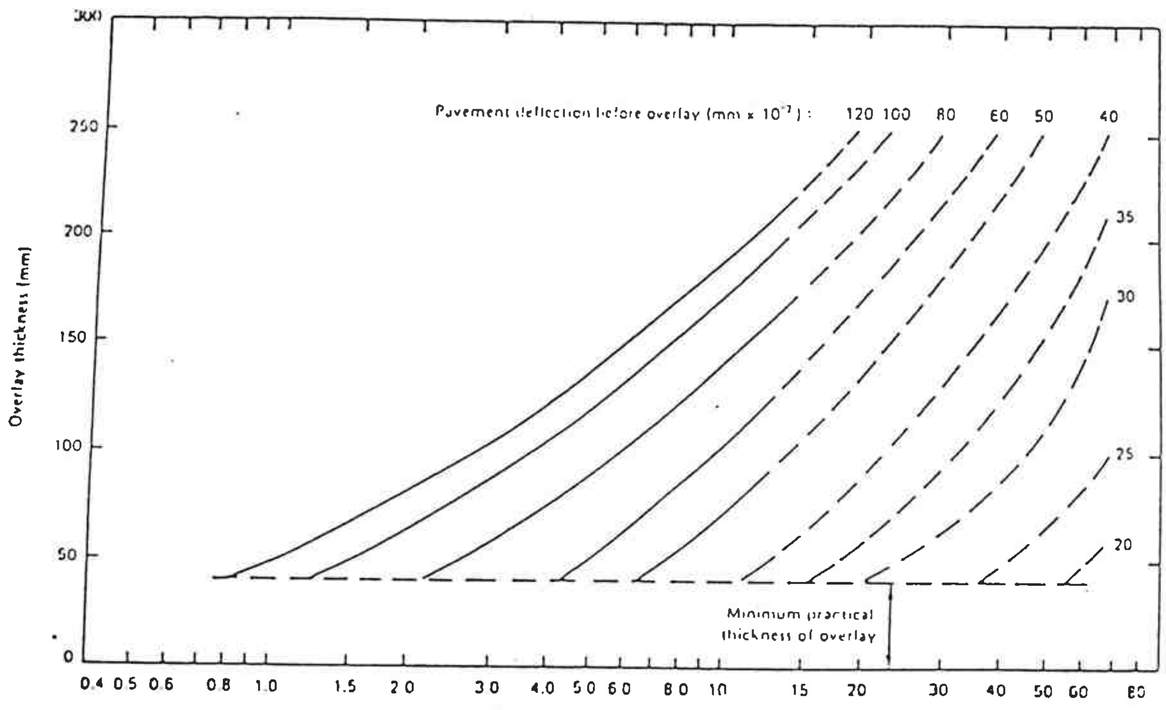
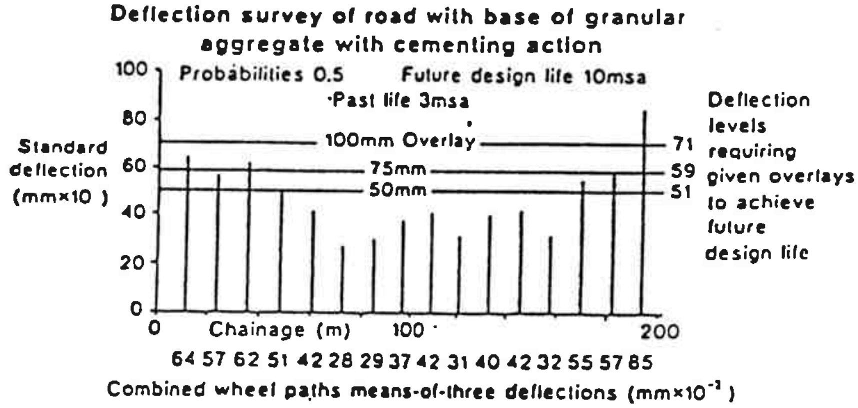


Figure A.12. Overlay Design Chart for Pavements with Granular Road Bases Whose Aggregates have a Natural Cementing Action (.90 Probability) (20).

Table A.2. Example of Detailed Overlay Design Before Application of Practical Restraints (20).



Section	0 to 100 m	100 to 200 m
Characteristic deflection	62 mm x 10^{-2}	57 mm x 10^{-2}
Initial overlay design for 10 msa future life	100 mm	75 mm
Maximum deflection to give 75% of 10 msa future life with initial overlay thickness	82 mm x 10^{-2}	69 mm x 10^{-2}
Any deflections above this level?	No	Yes, at 192 m
Any local reconstruction?	No	Yes, 186 to 200 m
Characteristic value of remaining deflections	--	55 mm x 10^{-2}
Revised overlay design	--	75 mm
Maximum deflection to give minimum life requirement when overlaid	--	69 mm x 10^{-2}
Any deflections above this level?	--	No
Final detailed overlay design	100 mm	100 to 186 m - 75 mm 186 m to 200 m - Local Reconstruction

values were significant in the overlay design. The multiple regression analysis produced the following relationship:

$$T = \frac{\log L + .104R + .000578P_o - .0653(SI)}{.0587\ (2.6 + 32D_5)^{.333}} \qquad (A.5)$$

where:

T = AC overlay thickness (in.),

L = expected traffic loading for design period (18k ESAL),

R = an environmental factor (AASHTO Regional Factor),

P_o = Mayes Ridemeter inches of roughness on existing pavement (in.),

D_5 = fifth dynaflect sensor deflection (mils), and

SI = spreadability index of existing pavement before AC overlay as determined below:

$$SI = \frac{D_1 + D_2 + D_3 + D_4 + D_5 \times 100}{5D_1} \qquad (A.6)$$

where:

$D_{1,2,3,4,5}$ = deflection measurements for 5 Dynaflect locations (mils).

The spreadability is a function of the modulus values for each layer. Arizona does not apply temperature correction factors to the deflections since their results prove the correction to be unreliable. This equation was the result of extensive research and it must be realized that it is only applicable for Arizona's conditions, special considerations and equipment.

A.2 Mechanistic Procedures

As in deflection-based procedures, nondestructive pavement evaluation condition surveys and traffic are required inputs. Figure A.13 shows a flowchart for the mechanistic design procedures. Condition surveys or a measure of distress and stiffness properties of the various materials are needed in this design approach. Distress types considered include permanent deformation and fatigue cracking. Stiffness may be determined through testing of representative samples or estimated from nondestructive measurements. Neither method should be used solely; it would be better to incorporate the data from both methods to find the layer stiffnesses. Table A.3 suggests a general guideline for field testing frequency. When the sections have been established, the design deflection is established. Representative material characteristics are determined from pavement cores, layer samples, thicknesses, and undisturbed subgrade samples.

Multilayer elastic analysis is used to estimate deflections under known loadings for a given laboratory-determined stiffness. These deflections are compared to the actual measured deflections; adjustments are then made to the stiffness values until the predicted and measured surface deflections are in reasonable agreement. Stiffness characteristics can also be estimated from surface deflection measurements. The shape of the deflected surface at various radii is used as input for a computer program to determine the modulus values which will give the best fit to the data.

Traffic considerations include not only the load equivalency concept previously discussed but also a distribution of traffic across the lanes and the concentrations of truck traffic in the outer lane. Remaining life is

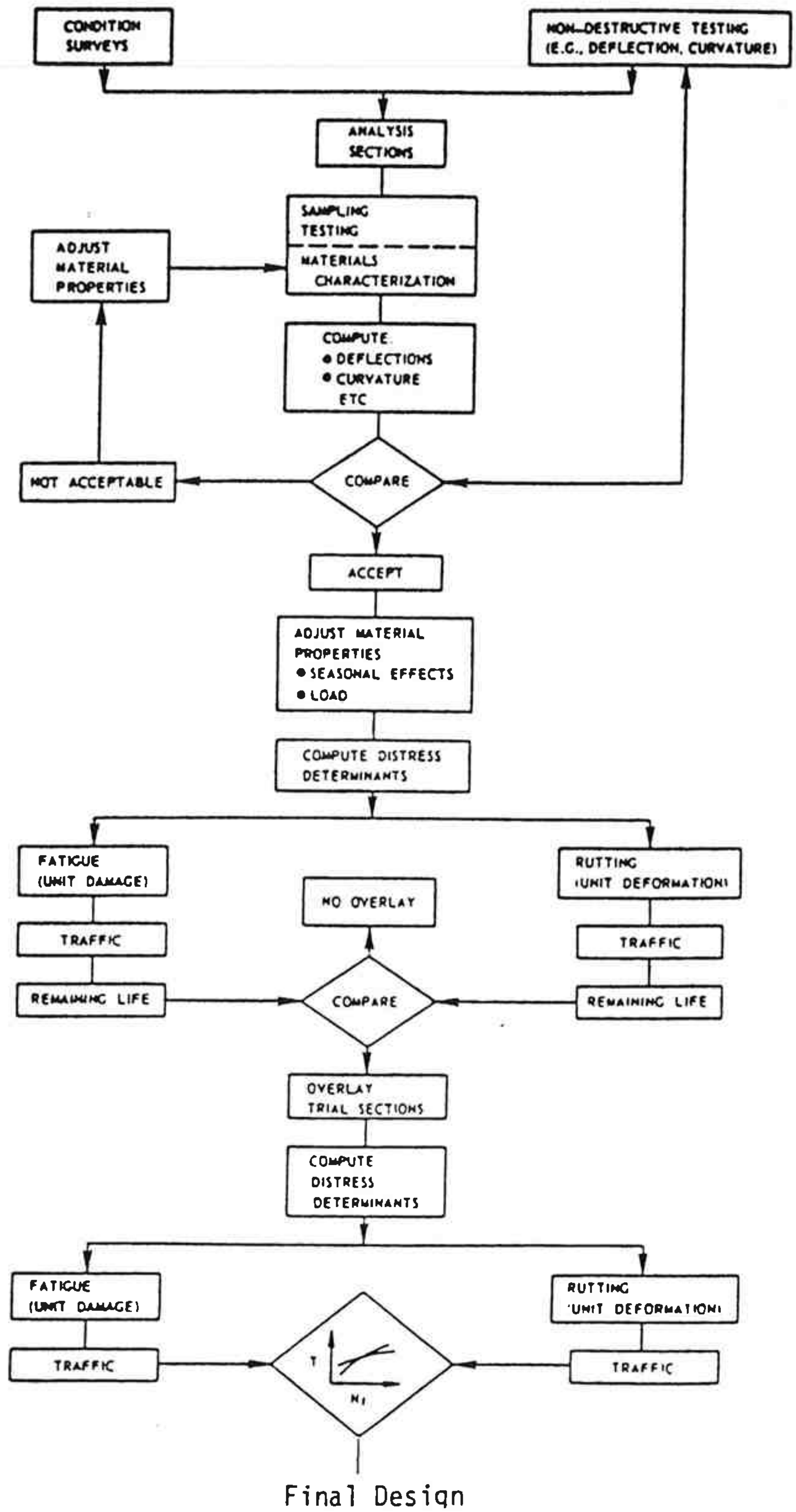


Figure A.13. Overlay Design Based on Analytical (Mechanistic) Analysis (17).

Table A.3. Suggested Spacing for Nondestructive Measurements.

Organization	Highways
Asphalt Institute (6)	20/mile (min.), 10/analysis section (min.)
Transport and Road Research Laboratory (20)	12 to 25 m
Shell Research (13)	25 to 50 m
Austin Research Engineers - FHWA (14)	100 to 250 ft depending on terrain and on material uniformity

computed from cumulative damage theory using a fatigue relationship based on strain:

$$N = A(1/e_t)^b(1/S_{mix})^c \tag{A.7}$$

where:

N = number of applications to failure,

e_t = tensile strain in asphalt concrete (in./in.),

S_{mix} = stiffness modulus of concrete (psi), and

A,b,c = constant for specific asphalt mix.

A simple form of the cumulative damage theory to determine remaining life is:

$$N_r/N_{D1} = 1 - N_{A1}/N_{D1} \tag{A.8}$$

where:

N_r/N_{D1} = remaining life,

N_{A1} = number of ESAL's to date,

N_{D1} = allowable number ESAL according to fatigue, and

N_r = additional 18k ESAL that can be applied to existing pavement.

From the tensile strain value and the allowable number of repetitions, it is possible to define a relationship between overlay thickness and additional load applications from a fatigue expression. Four procedures demonstrate the amount of input and the calculations necessary for determining overlay thicknesses using a mechanistic approach. In the following sections, these procedures are described briefly.

Shell Research Procedure (13). The structural response of the pavement is measured with the FWD. The maximum deflection as well as the shape of the deflected basin are used to determine the stiffness characteristics of each layer, defined by the modulus (E) and Poisson's ratio (μ). For the typical three-layer pavement, the modulus of the asphalt bound layer is estimated using a nomograph procedure. The thickness of the granular layer is estimated from construction reports or determined from coring. The subgrade modulus and the effective thickness of the asphalt concrete layer are determined using the iterative process until the predicted and measured values are approximately the same.

Distress in the form of excessive permanent deformation is limited by controlling the vertical compressive strain at the subgrade surface:

$$e_v = 2.8 \times 10^{-2} \times N^{-.25} \tag{A.9}$$

where:

e_v = compressive subgrade strain, and

N = number of load applications.

Fatigue cracking is controlled by limiting the tensile strain using:

$$N = A(1/e_t)^a(1/E_1)^b \tag{A.10}$$

where:

A,a,b = constant mixture parameters.

The remaining life is estimated from a knowledge of the layer thicknesses, design values, and applied traffic. If the design life of the existing section has been exceeded so that the section can not accommodate the anticipated traffic, an overlay thickness is required. This is accomplished by considering three separate thickness determinations: 1) thickness to satisfy the subgrade strain criteria; 2) thickness to satisfy fatigue strain in the asphalt layer; and 3) thickness assuming the existing pavement has deteriorated to the extent it acts as an unbound granular layer.

Once the modulus of the subgrade is determined, this value along with h_1 are used in Figure A.14 to find the original design life based on subgrade strain. This value is compared with the actual number of axle loads the pavement has seen to determine the remaining life. The additional number of axle loads is entered on Figure A.14 to ascertain the additional amount of asphalt needed to satisfy the subgrade criteria.

Using a similar procedure for condition 2 (fatigue strain) and employing the use of the equation below to find the design number, the thickness to preclude fatigue is determined and compared to the previous one to determine which condition should govern:

$$N_{D2} = N_{D1} N_{A2} / N_{D1} - N_{A1} \tag{A.11}$$

where:

N_{A2} = number of additional standard axles anticipated,

N_{D1} = original design life based on asphalt criteria, and

N_{A1} = actual number of repetitions at the present.

Federal Highway Administration-ARE (14). Any type of deflection equipment may be used in this procedure to obtain deflections. The frequency of deflections depend on the terrain as shown in Table A.4. Condition surveys are performed at the same time the deflection measurements are made. Deflection profiles are generated for the entire length of the project to assist in establishing the analysis sections. The stiffness modulus and Poisson's ratio are determined for each layer from laboratory tests on cores or remolded samples.

Asphalt concrete is assigned a modulus of 70,000 psi if it is cracked, otherwise it should be determined over a range of temperatures corresponding to a loading time which simulates traffic. Fatigue life is defined in terms

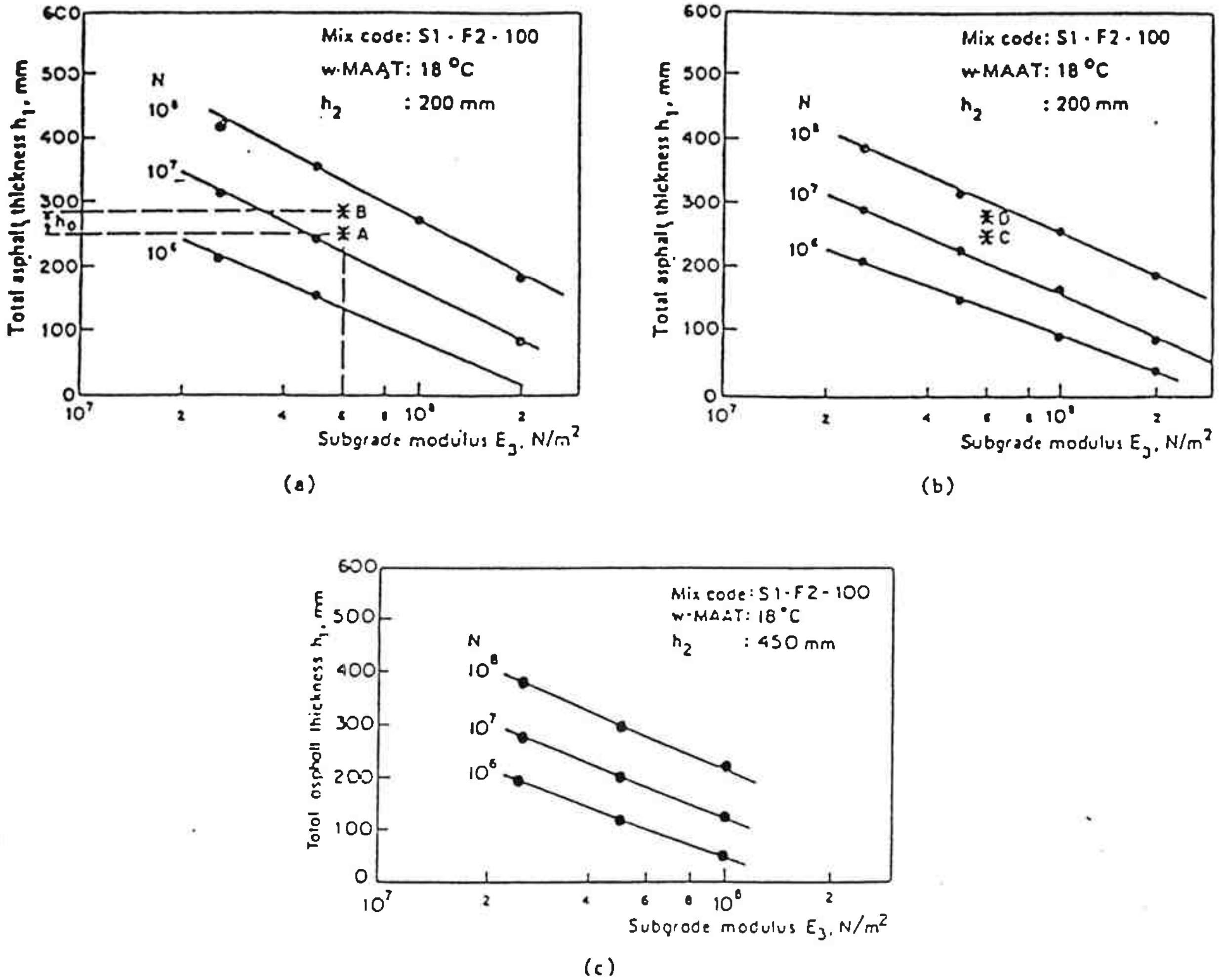


Figure A.14. Interpolated Design Charts: (a) Based on Subgrade Strain Criterion, (b) Based on Asphalt Strain Criterion, and (c) Based on the Assumption that the Existing Asphalt Concrete Behaves as Granular Base (13).

Table A.4. Guidelines for Deflection Measurements (14).

Type of Location	Spacing (ft)
Rolling terrain	100
Numerous cut-to-fill transitions	100
Level with uniform grading	250

(1 ft = .3048 m)

of magnitude of the tensile strain (e_t) determined from ELSYM5 computer program and inserted into the following equation to calculate the expected number of load repetitions to failure:

$$N = 9.73 \times 10^{-15} (1/e_t)^{5.16} \quad \text{(A.12)}$$

where:

N = allowable number of 18k ESAL, and

e_t = horizontal tensile strain on the underside asphalt bound layer.

Rutting is controlled by the following equation:

$$N = 1.365 \times 10^{-9} (\epsilon_c)^{-4.477} \quad \text{(A.13)}$$

where:

N = allowable number of 18k ESAL, and

ϵ_c = vertical compressive strain on top subgrade layer.

The remaining life is determined for an uncracked pavement from an estimation of the amount of traffic (18k ESAL) applied to date, and the tensile strain on the underside of the asphalt bound layer. The ELSYM computer program can be used to compute the strain. Knowing the strains, modulus values, and Poisson's ratio for the various layers, the total number of load repetitions may be found from Eq. (A.13). The remaining life is then calculated using the following equation.

$$N_r = N_1(1 - N_D/N_1) \quad \text{(A.14)}$$

where:

N_r = number of additional 18k ESAL for computed strain level,

N_D = design traffic volume,

N_1 = total number of load repetitions (from Eq. A.13).

If N_r is less than the expected traffic then an overlay is required. A typical relationship between fatigue and rutting versus equivalent load applications is shown on Figure A.15. The value of N for each is used to determine the overlay thickness depending upon the amount of cracking. Computer solutions are available to solve this.

FHWA-RII (25). This method, an extension of the ARE approach, only considers fatigue cracking for the design process. Deflections are obtained from the Dynaflect, Road Rater, or FWD. The shape of the deflected pavement surface is defined by at least four points. Analysis sections are not used; the deflection data from each testing location is analyzed separately. Multilayer elastic structures of either 3 or 4 layers are employed in this design. The layer moduli is determined using ELSYM and the following inputs:

1) Surface deflection measurements,
2) Base type,
3) Layer thicknesses,
4) Poisson's ratio for each layer, and
5) Modulus of asphalt concrete at test temperature.

Nonlinear response characteristics of granular layers are represented by the equation:

$$M_r = kS^n \qquad \text{(A.15)}$$

where:

S = sum of principle stresses (or deviator stress for fine-grained soils), and

k,n = material coefficients.

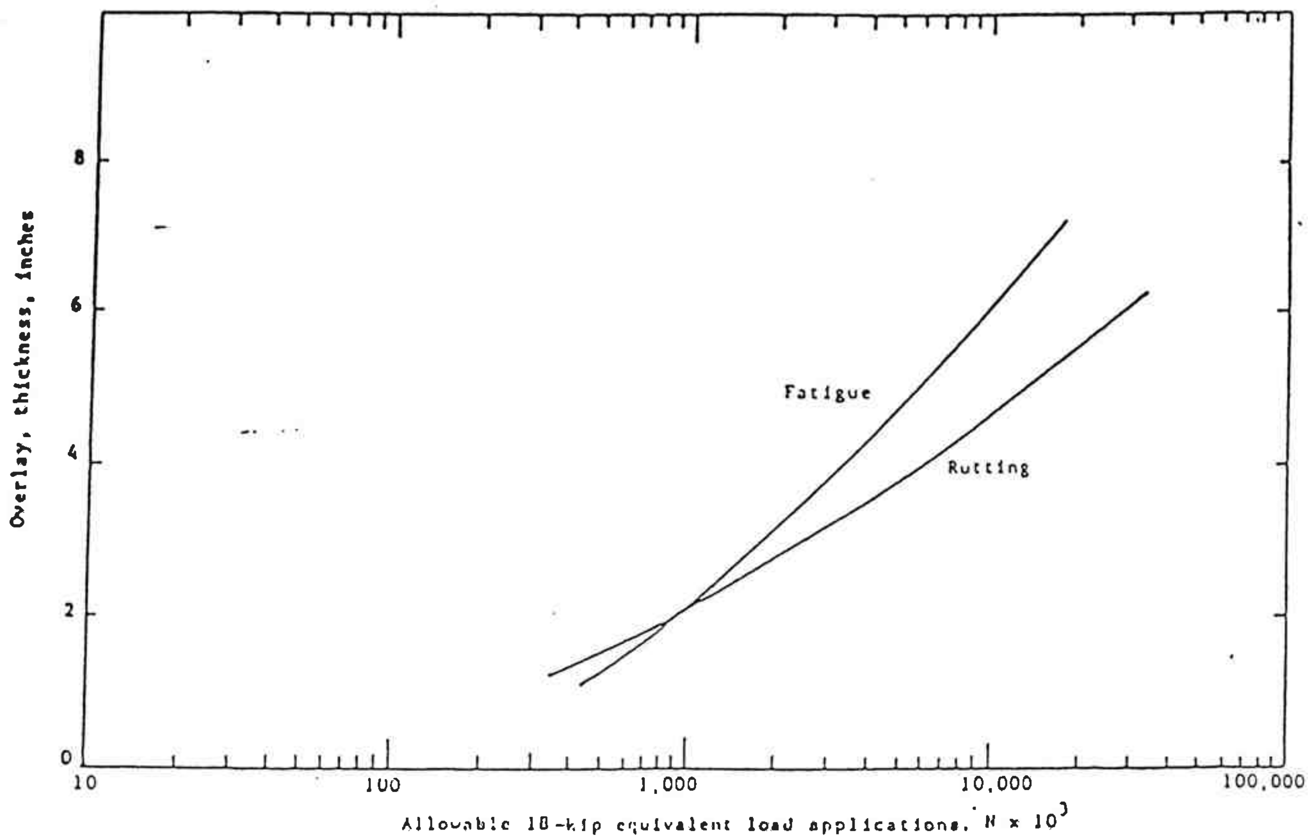


Figure A.15. Sample Overlay Thickness Design Curves (14).

The relationship between load applications and strain was developed from AASHTO Road Test data

$$N_{D1} = 7.56 \times 10^{-12} (1/e_t)^{4.68} \tag{A.16}$$

where:

N_{D1} = number of load applications, and

e_t = maximum tensile strain parallel to the direction of traffic.

The number of repetitions to date is adjusted using a regional and seasonal factor. This value along with the total number of repetitions determined from the equation above determines the remaining life as follows:

$$N_r/N_{D1} = 1 - N_{A1}/N_{D1} \tag{A.17}$$

where:

N_r = number of additional 18k ESAL's for computed strain level

N_{D1} = calculated in Eq. (A.17)

N_{A1} = actual traffic applied to date.

If an overlay is required, it is calculated using the resulting strain from additional thicknesses of asphalt concrete. This procedure is repeated for each location. An evaluation of all the resulting thicknesses will provide a range of values to be used. The design is usually based on the 95th percentile of thickness.

New Mexico State Highway Department (42). Deflection measurements are obtained from a Road Rater device for each mile of project length. The deflections are adjusted for temperature conditions as shown below:

$$F_{70} = 29.56T_{AC}^{-.7935} \quad (E_{subgrade} > 3500 \text{ psi}) \tag{A.18}$$

$$F_{70} = 194.15T_{AC}^{-1.231} \quad (E_{subgrade} < 3500 \text{ psi}) \tag{A.19}$$

where:

F_{70} = correction factor multiplied to standard deflection for normalization.

$$T_{AC} = (.7077T - 58.9)D_{AC}H - .138 + 42.8 \tag{A.20}$$

where:

T_{AC} = average asphalt temperature (°F),

T = measured surface temperature plus the average air temperature for the preceding five days, and

D_{AC} = depth of the existing AC layer.

These standardized deflection values are the input into NMSHD computer program for backcalculating layer moduli. The overall layer moduli for the one-mile section is determined as shown:

$$LE_i = E_i - .7(SD) \tag{A.21}$$

where:

LE_i = elastic modulus for the layer,

E_i = mean elastic modulus, and

SD = standard deviation.

This procedure is repeated for each mile of project length. The design engineer determines whether some sections should be designed separately due to extensive damage. The resilient modulus of the asphalt cement concrete for the desired overlay is then determined by one of these methods:

1) coring a completed construction project with the same aggregate and asphalt source.

2) testing of laboratory samples with the same aggregate and asphalt source.

3) an empirical equation which is a function of the fines content, percent air voids, asphalt content, temperature, viscosity, and penetration of the asphalt cement.

The traffic is all converted to 18k ESAL. Estimates of prior traffic, future traffic, and allowable traffic are employed to determine the remaining life of the existing pavement.

The remaining life factors as well as the modulus values for the in situ pavement and the proposed overlay are all input into the modified DAMA computer program which operates on the fatigue and compressive strain criteria for failure of the pavement structure. An initial thickness for the overlay is assumed. Iteration of this value takes place until the desired design life criteria is met. Then that specific assumed thickness becomes the overlay design thickness.

APPENDIX B

DETAILED PROJECT DATA

The traffic history for the three projects evaluated are given in Figures B.1 and B.2. Traffic data was obtained from ODOT. Plots of the maximum deflection data recorded by each type of NDT equipment are shown on Figures B.3 through B.5 for each testing location. The Dynaflect readings were converted to Benkelman Beam readings. Raw data for the other sensor locations are contained in Tables B.1 through B.3.

Table B.4 shows an equivalent area for Dynaflect load for use with BISDEF. This was accomplished by creating a carbon paper print upon loading. Cores were taken from each project to determine the surface modulus of each pavement section. The bulk specific gravity, ASTM 2726, (Table B.5) and diametral resilient modulus, ASTM 4123, tests were performed (Table B.6).

The modulus of the portland cement concrete was obtained with use of resistive strain gages. The 4-in. diameter cylinders were tested in compression with strain gages attached as shown in Figure B.6. A strain meter detects the change in strain from the change in resistance of the wire gages. A full-bridge configuration was set up with active and compensating gages. The strain was recorded for several stress levels as shown in Table B.7.

The laboratory determined surface modulus was input as fixed values into the BISDEF program. The BISDEF program then calculated the other pavement layer moduli. Tables B.8 through B.10 summarize the calculations for the FWD. These values can be compared with the FWD BISDEF determined layer moduli without using the laboratory determined surface modulus (Tables B.11 through B.13). The Dynaflect deflection data was also input into BISDEF using the

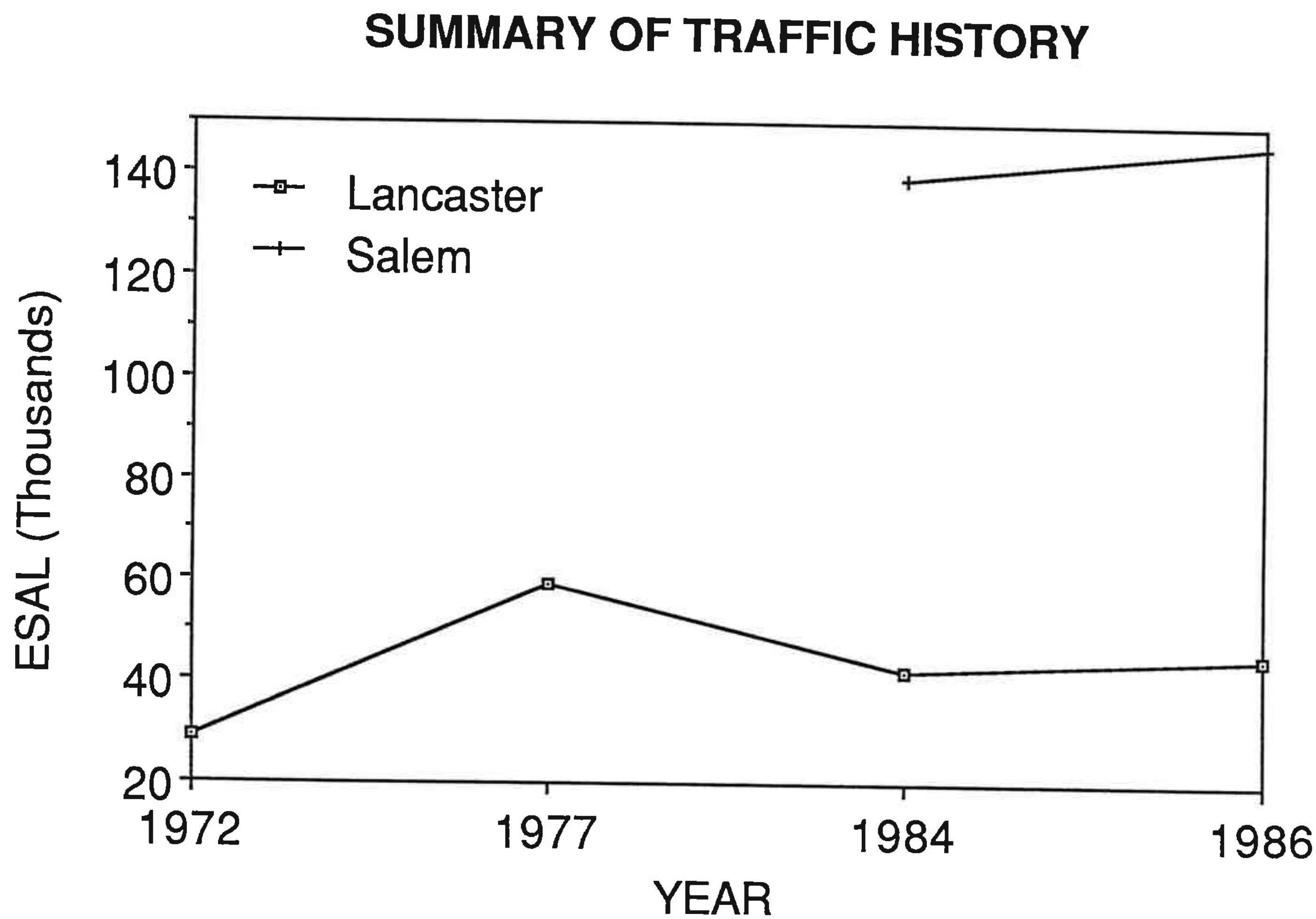


Figure B.1. Summary of Traffic History for Lancaster and Salem Parkway.

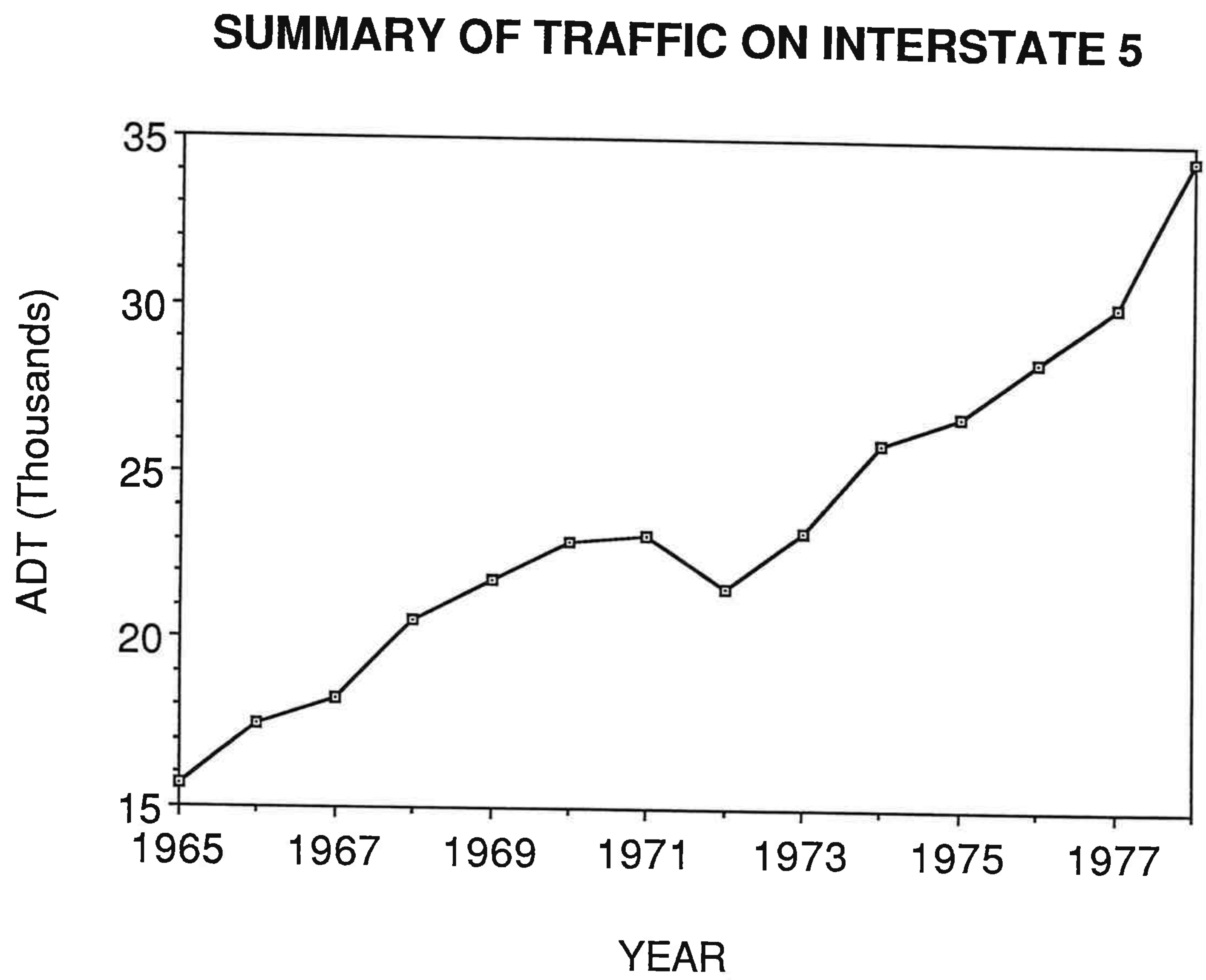


Figure B.2. Traffic History for Interstate 5.

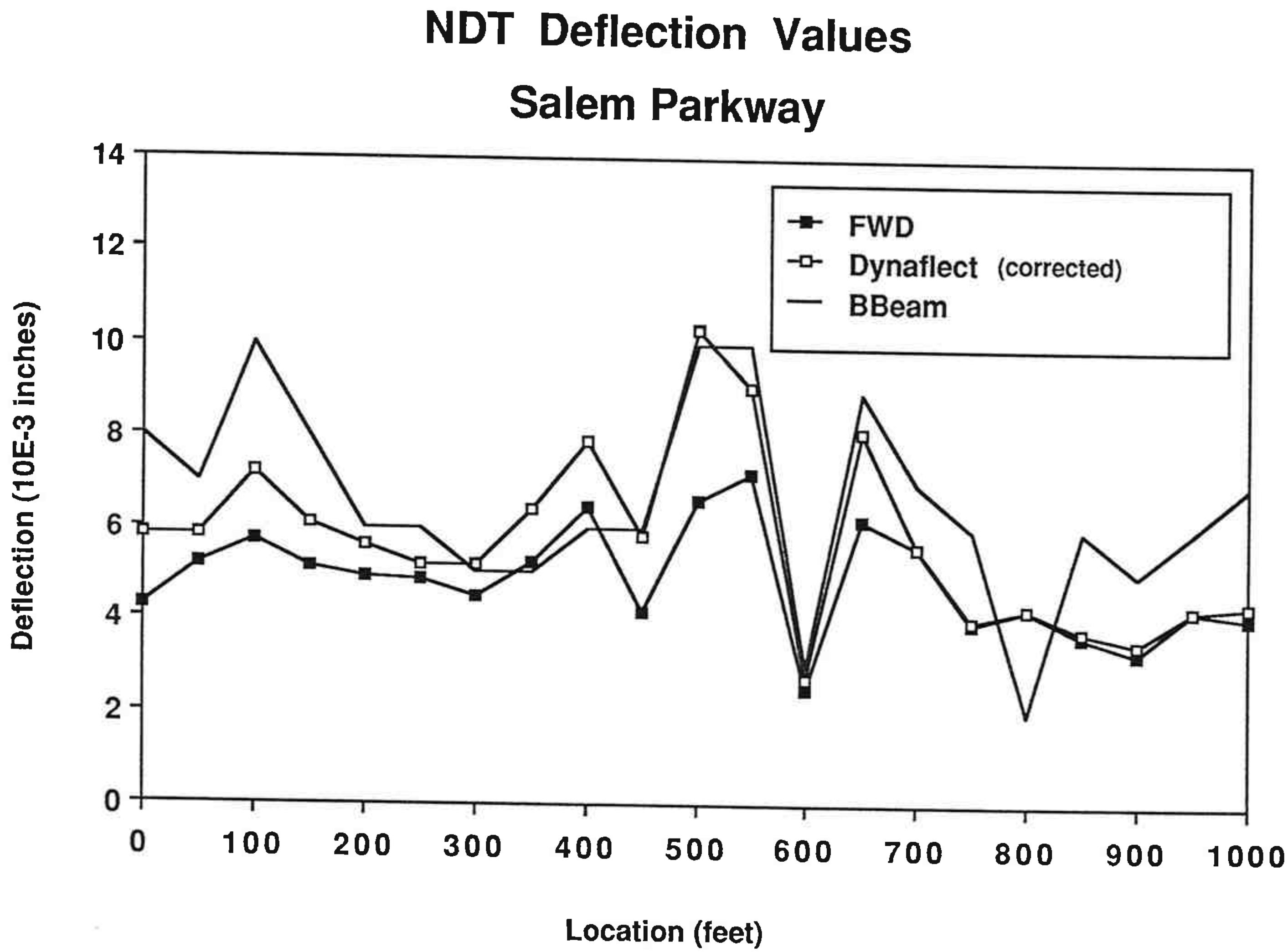


Figure B.3. Deflection Variation for NDT Equipment (Salem Parkway).

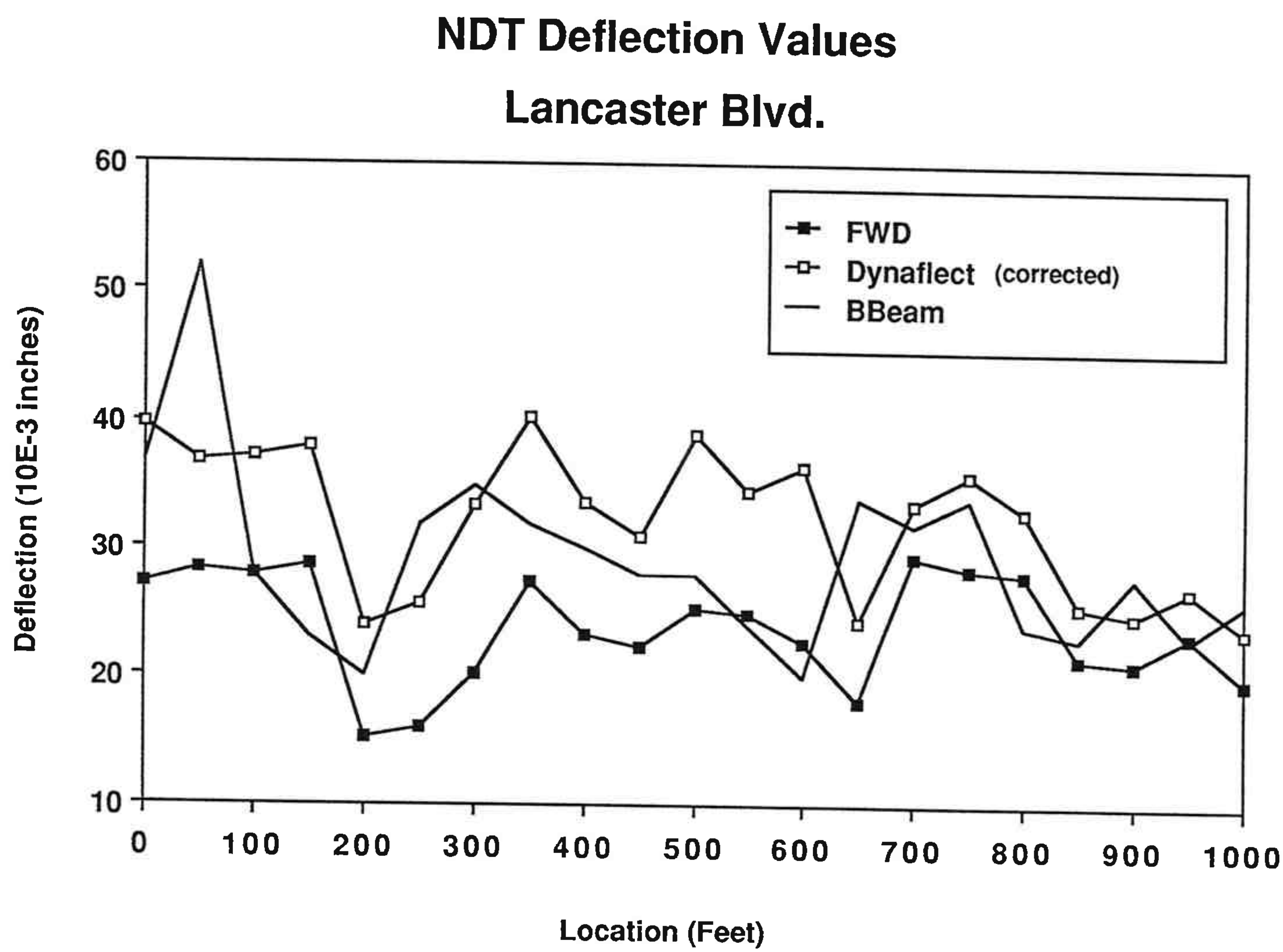


Figure B.4. Deflection Variation for NDT Equipment (Lancaster Blvd.).

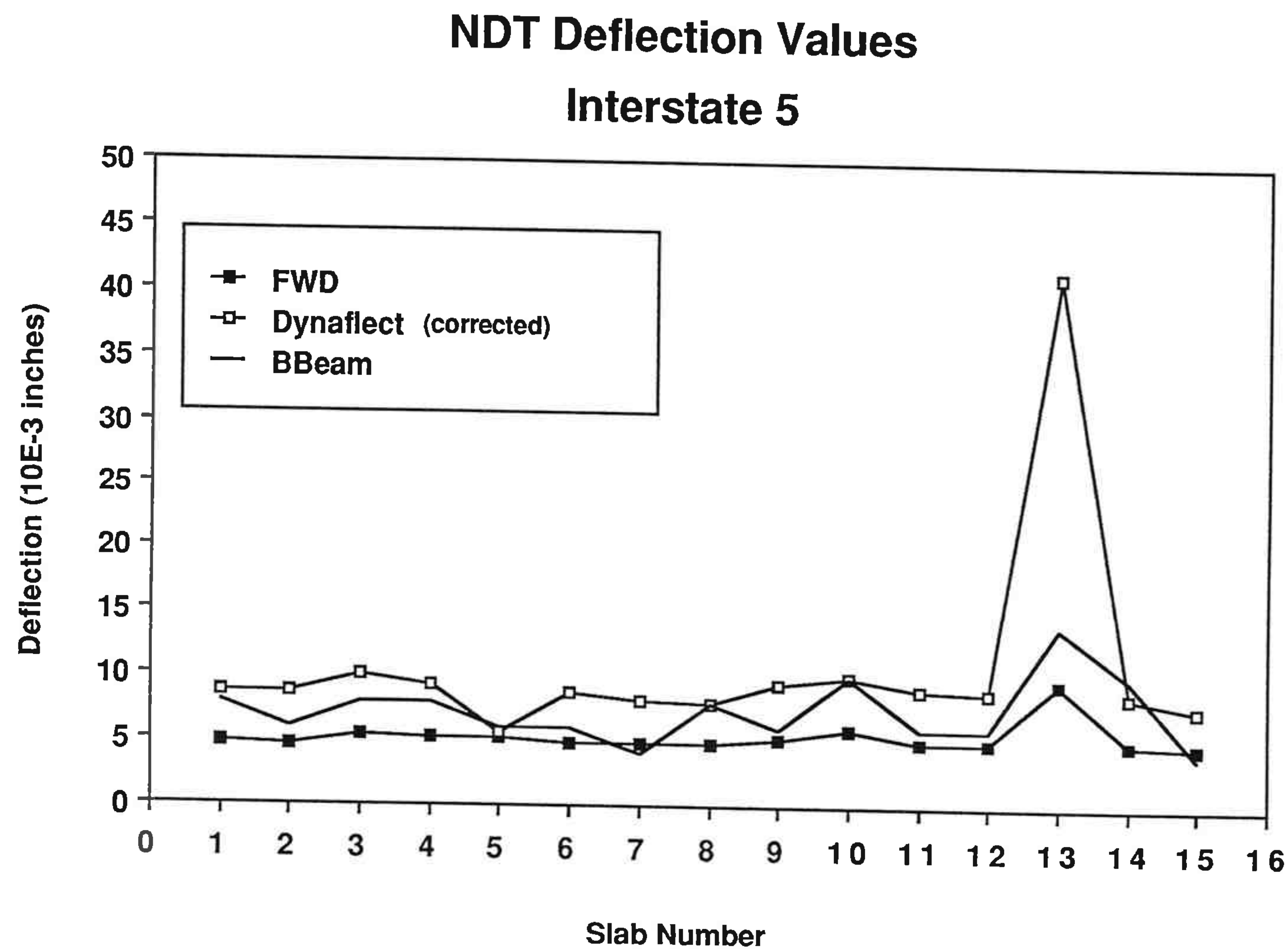


Figure B.5. Deflection Variation for NDT Equipment (Interstate 5).

Table B.1. Salem Parkway Deflection Values (Unadjusted for Temperature, $x10^{-3}$ in.).

Reading Number	Equipment	Load (lbs)	Sensors #1		#2	#3	#4
1	FWD	9,000	4.25		3.60	3.60	2.90
	Dynaflect	1,000	0.48	(5.8)*	0.44	0.34	0.28
	BBeam	9,000	8.00				
2	FWD	9,000	5.18		3.95	3.20	2.64
	Dynaflect	1,000	0.48	(5.8)	0.44	0.36	0.29
	BBeam	9,000	7.00				
3	FWD	9,000	5.72		4.69	3.39	2.82
	Dynaflect	1,000	0.57	(7.2)	0.48	0.40	0.33
	BBeam	9,000	10.00				
4	FWD	9,000	5.13		4.13	3.36	2.66
	Dynaflect	1,000	0.50	(6.1)	0.44	0.35	0.28
	BBeam	9,000	8.00				
5	FWD	9,000	4.90		3.83	3.19	2.56
	Dynaflect	1,000	0.47	(5.6)	0.43	0.35	0.28
	BBeam	9,000	6.00				
6	FWD	9,000	4.86		3.91	3.10	2.67
	Dynaflect	1,000	0.44	(5.2)	0.40	0.34	0.29
	BBeam	9,000	6.00				
7	FWD	9,000	4.51		3.85	2.94	2.61
	Dynaflect	1,000	0.44	(5.2)	0.40	0.34	0.29
	BBeam	9,000	5.00				
8	FWD	9,000	5.22		4.51	3.34	2.85
	Dynaflect	1,000	0.52	(6.4)	0.48	0.40	0.33
	BBeam	9,000	5.00				
9	FWD	9,000	6.46		4.92	3.88	3.42
	Dynaflect	1,000	0.61	(7.9)	0.58	0.47	0.38
	BBeam	9,000	6.00				
10	FWD	9,000	4.18		3.52	2.92	2.78
	Dynaflect	1,000	0.48	(5.8)	0.45	0.39	0.34
	BBeam	9,000	6.00				
11	FWD	9,000	6.64		5.43	4.37	3.89
	Dynaflect	1,000	0.75	(10.3)	0.71	0.63	0.55
	BBeam	9,000	10.00				
12	FWD	9,000	7.21		5.51	4.36	3.57
	Dynaflect	1,000	0.68	(9.1)	0.61	0.48	0.37
	BBeam	9,000	10.00				
13	FWD	9,000	2.53		1.80	1.56	1.58
	Dynaflect	1,000	0.27	(2.7)	0.25	0.23	0.21
	BBeam	9,000	3.00				
14	FWD	9,000	6.22		4.89	3.86	3.23
	Dynaflect	1,000	0.62	(8.1)	0.52	0.42	0.35
	BBeam	9,000	9.00				
15	FWD	9,000	5.61		4.43	3.51	2.80
	Dynaflect	1,000	0.47	(5.6)	0.42	0.35	0.28
	BBeam	9,000	7.00				
16	FWD	9,000	3.95		2.95	2.32	1.94
	Dynaflect	1,000	0.36	(4.0)	0.32	0.27	0.21
	BBeam	9,000	6.00				
17	FWD	9,000	4.25		3.32	2.79	2.23
	Dynaflect	1,000	0.38	(4.3)	0.33	0.28	0.23
	BBeam	9,000	2.00				
18	FWD	9,000	3.68		2.99	2.63	2.32
	Dynaflect	1,000	0.35	(3.8)	0.32	0.28	0.23
	BBeam	9,000	6.00				
19	FWD	9,000	3.32		2.88	2.44	2.25
	Dynaflect	1,000	0.33	(3.5)	0.31	0.27	0.23
	BBeam	9,000	5.00				
20	FWD	9,000	4.28		3.09	2.66	2.15
	Dynaflect	1,000	0.38	(4.3)	0.32	0.27	0.22
	BBeam	9,000	6.00				
21	FWD	9,000	4.13		3.39	2.76	2.41
	Dynaflect	1,000	0.39	(4.4)	0.35	0.30	0.25
	BBeam	9,000	7.00				

(1 lb = 4.448 N; 1 in. = 2.54 cm)
*Converted to equivalent Benkelman Beam value using Fig. 2.1

Table B.2. Lancaster Deflection Values (Unadjusted for Temperature, $x10^{-3}$ in.).

Reading Number	Equipment	Load (lbs)	Sensors #1	#2	#3	#4
1	FWD	9,000	27.24	16.14	7.66	4.62
	Dynaflect	1,000	2.12 (39.8)*	1.25	0.69	0.45
	BBeam	9,000	37.00			
2	FWD	9,000	28.40	17.34	8.43	5.08
	Dynaflect	1,000	2.00 (36.9)	1.23	0.69	0.47
	BBeam	9,000	52.00			
3	FWD	9,000	28.02	16.87	7.95	4.68
	Dynaflect	1,000	2.01 (37.2)	1.22	0.67	0.44
	BBeam	9,000	28.00			
4	FWD	9,000	28.78	18.29	9.22	5.01
	Dynaflect	1,000	2.05 (38.1)	1.39	0.75	0.46
	BBeam	9,000	23.00			
5	FWD	9,000	15.22	10.44	6.02	3.88
	Dynaflect	1,000	1.43 (23.9)	1.06	0.64	0.43
	BBeam	9,000	20.00			
6	FWD	9,000	15.92	3.91	3.10	2.67
	Dynaflect	1,000	1.51 (25.6)	1.16	0.74	0.49
	BBeam	9,000	32.00			
7	FWD	9,000	20.05	13.18	7.26	4.48
	Dynaflect	1,000	1.85 (33.4)	1.26	0.72	0.47
	BBeam	9,000	35.00			
8	FWD	9,000	27.30	16.75	7.96	4.95
	Dynaflect	1,000	2.14 (40.3)	1.34	0.78	0.49
	BBeam	9,000	32.00			
9	FWD	9,000	23.10	14.59	7.99	4.83
	Dynaflect	1,000	1.86 (33.6)	1.24	0.71	0.45
	BBeam	9,000	30.00			
10	FWD	9,000	22.22	14.55	7.70	4.93
	Dynaflect	1,000	1.75 (31.0)	1.26	0.68	0.45
	BBeam	9,000	28.00			
11	FWD	9,000	25.23	15.73	8.43	4.77
	Dynaflect	1,000	2.09 (39.1)	1.36	0.72	0.46
	BBeam	9,000	28.00			
12	FWD	9,000	24.98	16.01	8.35	4.77
	Dynaflect	1,000	1.90 (34.6)	1.29	0.74	0.46
	BBeam	9,000	24.00			
13	FWD	9,000	22.67	13.49	7.15	4.62
	Dynaflect	1,000	1.77 (31.5)	1.77	1.11	0.45
	BBeam	9,000	20.00			
14	FWD	9,000	18.07	11.92	6.74	4.49
	Dynaflect	1,000	1.45 (24.3)	1.07	0.69	0.45
	BBeam	9,000	34.00			
15	FWD	9,000	29.39	18.97	9.87	5.89
	Dynaflect	1,000	1.86 (33.6)	1.24	0.72	0.46
	BBeam	9,000	32.00			
16	FWD	9,000	28.49	18.24	8.72	5.05
	Dynaflect	1,000	1.98 (36.5)	1.26	0.65	0.37
	BBeam	9,000	34.00			
17	FWD	9,000	28.16	18.40	9.44	6.03
	Dynaflect	1,000	1.84 (33.1)	1.22	0.69	0.49
	BBeam	9,000	24.00			
18	FWD	9,000	21.39	13.33	7.09	4.72
	Dynaflect	1,000	1.51 (25.6)	1.06	0.68	0.48
	BBeam	9,000	23.00			
19	FWD	9,000	20.99	13.33	7.09	4.72
	Dynaflect	1,000	1.47 (24.8)	1.01	0.62	0.45
	BBeam	9,000	28.00			
20	FWD	9,000	23.32	15.38	7.96	5.02
	Dynaflect	1,000	1.57 (27.0)	1.08	0.63	0.44
	BBeam	9,000	23.00			
21	FWD	9,000	19.77	12.56	7.19	4.87
	Dynaflect	1,000	1.42 (23.7)	0.98	0.62	0.46
	BBeam	9,000	26.00			

(1 lb = 4.448 N; 1 in. = 2.54 cm)
*Converted to equivalent Benkelman Beam value using Fig. 2.1

Table B.3. Interstate 5 Deflection Values (Unadjusted for Temperature, $x10^{-3}$ in.).

Reading Number	Equipment	Load (lbs)	Sensors #1	#2	#3	#4
	FWD	9,000	4.92	4.37	3.52	2.75
1	Dynaflect	1,000	0.65 (8.6)*	0.61	0.51	0.41
	BBeam	9,000	8.00			
	FWD	9,000	4.68	4.05	3.23	2.82
2	Dynaflect	1,000	0.65 (8.6)	0.61	0.51	0.42
	BBeam	9,000	6.00			
	FWD	9,000	5.32	4.77	3.88	3.22
3	Dynaflect	1,000	0.73 (10.0)	0.69	0.57	0.47
	BBeam	9,000	8.00			
	FWD	9,000	5.22	4.60	3.64	3.30
4	Dynaflect	1,000	0.69 (9.3)	0.63	0.53	0.44
	BBeam	9,000	8.00			
	FWD	9,000	5.17	4.56	3.55	3.25
5	Dynaflect	1,000	0.47 (5.6)	0.63	0.53	0.44
	BBeam	9,000	6.00			
	FWD	9,000	4.81	4.40	3.42	3.03
6	Dynaflect	1,000	0.65 (8.6)	0.60	0.49	0.40
	BBeam	9,000	6.00			
	FWD	9,000	4.76	4.31	3.73	2.92
7	Dynaflect	1,000	0.62 (8.1)	0.58	0.48	0.40
	BBeam	9,000	4.00			
	FWD	9,000	4.79	4.36	3.33	2.90
8	Dynaflect	1,000	0.61 (7.9)	0.58	0.49	0.41
	BBeam	9,000	8.00			
	FWD	9,000	5.13	4.83	3.80	3.47
9	Dynaflect	1,000	0.70 (9.4)	0.65	0.54	0.45
	BBeam	9,000	6.00			
	FWD	9,000	5.89	5.17	4.21	3.40
10	Dynaflect	1,000	0.73 (10.0)	0.69	0.58	0.48
	BBeam	9,000	10.00			
	FWD	9,000	5.03	4.75	3.46	3.41
11	Dynaflect	1,000	0.68 (9.1)	0.64	0.54	0.45
	BBeam	9,000	6.00			
	FWD	9,000	5.02	4.61	3.75	3.17
12	Dynaflect	1,000	0.67 (8.9)	0.63	0.52	0.43
	BBeam	9,000	6.00			
	FWD	9,000	9.59	10.31	3.80	2.86
13	Dynaflect	1,000	2.18 (41.3)	1.85	1.43	1.02
	BBeam	9,000	14.00			
	FWD	9,000	4.93	4.51	3.51	2.92
14	Dynaflect	1,000	0.65 (8.6)	0.60	0.50	0.40
	BBeam	9,000	10.00			
	FWD	9,000	4.75	4.27	3.44	2.79
15	Dynaflect	1,000	0.60 (7.7)	0.56	0.45	0.38
	BBeam	9,000	4.00			

(1 lb = 4.448 N; 1 in. = 2.54 cm)
*Converted to equivalent Benkelman Beam value using Fig. 2.1

Table B.4. Converted Dynaflect Area.

	Length (in.)	Width (in.)	Footprint Area (in.^{2})	Equivalent Radius (in.)	Load (lbs)	Pressure (psi)
PCC Footprint (1 wheel)	.75	3.5	2.625	.914	500	190.5
AC Footprint (1 wheel)	.875	3.5	3.062	.987	500	163.3

(1 in. = 2.54 cm; 1 lb = 4.448 N; 1 psi = 6,895 N/m^2)

Table B.5. Bulk Specific Gravity Determination.

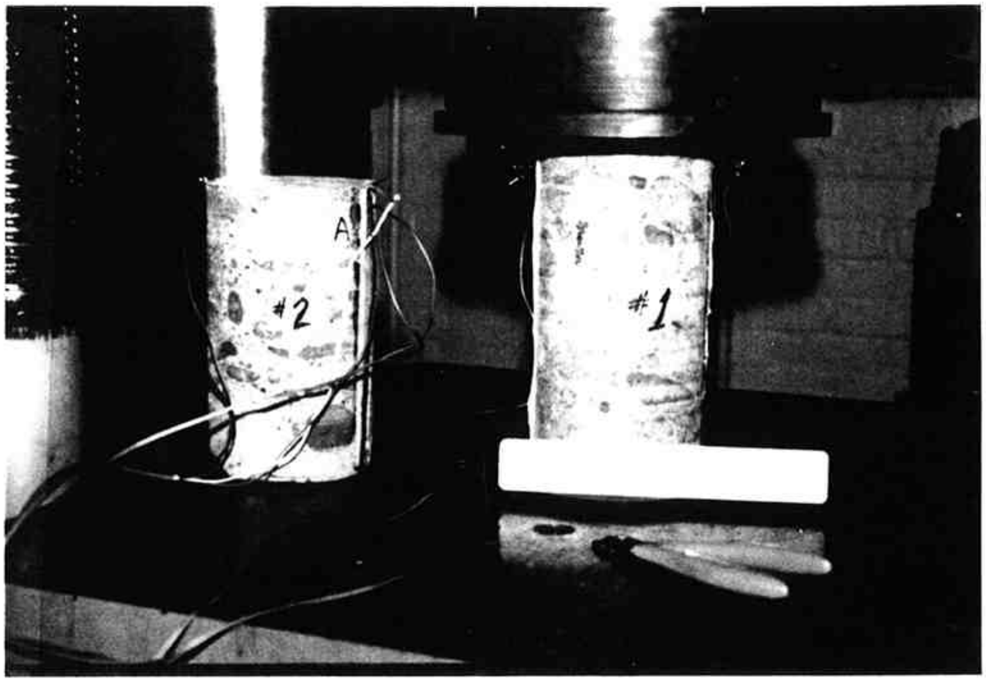
Sample Location	Air Wt. (gms)	Immersed Wt. (gms)	SSD Wt. (gms)	Specific* Gravity
Salem 1 (AC)	1186.2	609.7	1091.4	2.463
Salem 2 (AC)	1189.0	612.3	1094.2	2.467
Salem 3 (AC)	1172.3	662.3	1174.9	2.287
Salem 1 (CTB)	973.2	556.9	988.8	2.253
Salem 2 (CTB)	942.1	544.5	961.0	2.262
Lancaster 1 (AC)	1239.9	712.1	1240.7	2.346
Lancaster 2 (AC)	1197.3	670.0	1202.9	2.247

$$*\text{Specific Gravity} = \frac{\text{Air Wt.}}{\text{SSD Wt.} - \text{Immersed Wt.}}$$

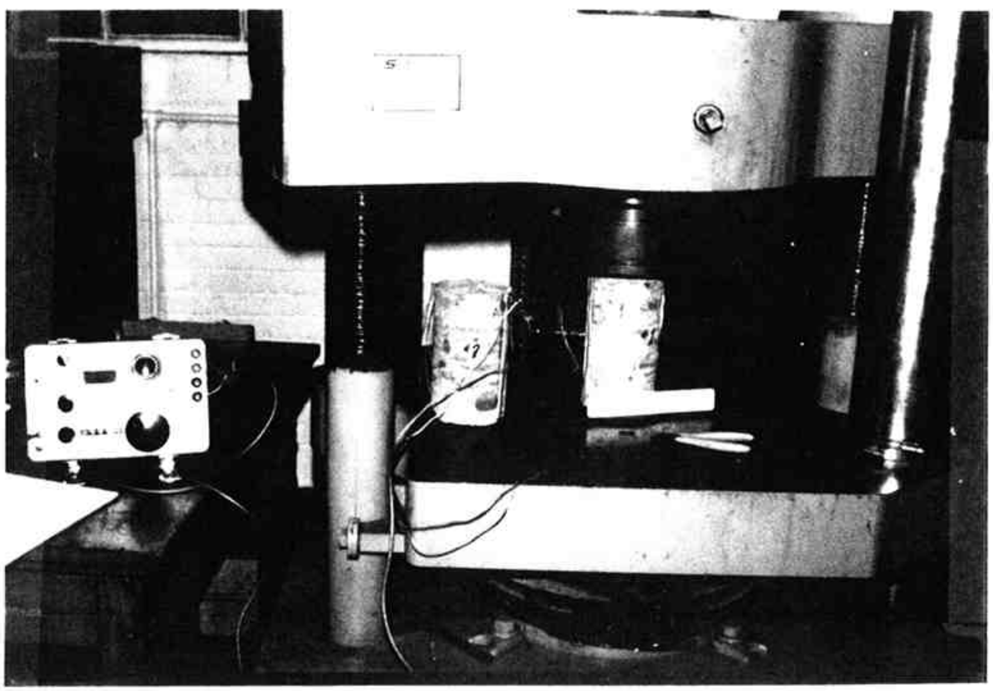
Table B.6. Resilient Modulus Values from Cores (at 23°C).

Sample Location	Thickness (in.)	Load (lbs)	Strain ($x10^6$)	Modulus (psi)
Salem 1 (AC)	2.343	304.3	91.1	$8.81x10^5$
Salem 2 (AC)	2.373	437.4	125.0	$9.12x10^5$
Salem 3 (AC)	2.507	437.4	135.7	$7.95x10^5$
			Average	$8.63x10^5$
Salem 1 (CTB)	2.090	697.9	67.0	$3.08x10^6$
Salem 2 (CTB)	2.143	697.9	136.6	$1.47x10^6$
			Average	$2.28x10^6$
Lancaster 1 (AC)	2.543	437.4	60.7	$1.75x10^6$
Lancaster 2 (AC)	2.600	437.4	67.0	$1.53x10^6$
			Average	$1.65x10^6$

(1 in. = 2.54 cm; 1 lb = 4.448 N; 1 psi = 6,895 N/m^2)



(a) Test Cylinders



(b) Strain Meter

Figure B.6. Testing Apparatus for PCC Modulus.

Table B.7. Modulus Values for PCC Cores.

Sample Location	Load (lbs)	Area (in.^{2})	Strain	Modulus (psi)
Interstate 5-1 (PCC)	500	12.57	8	4,976,000
	1,000		20.5	3,884,000
	1,500		32.5	3,675,000
	2,000		46.5	3,424,000
	2,500		60	3,317,000
			Average	3,885,000
Interstate 5-2 (PCC)	500		2.3	1,731,000
	1,000		38.5	2,068,000
	1,500		53	2,253,000
	2,000		66.5	2,394,000
	2,500		80	2,488,000
			Average	2,187,000
Interstate 5-3 (PCC)	500		17.5	2,275,000
	1,000		33	2,413,000
	1,500		46	2,596,000
	2,000		58	2,745,000
	2,500		71	2,803,000
			Average	2,566,000
		Project Length Average		2,869,000

(1 lb = 4.448 N; 1 in.^{2} = 6.45 cm^2; 1 psi = 6,895 N/m^2)

Table B.8. Modulus Values for Salem Parkway Pavement Structure Calculated from FWD Deflection Basin (BISDEF with AC Core Samples).

Deflection Location	AC (psi)	CTB (psi)	CTS (psi)	Subgrade (psi)
2	863,000	679,705	55,260	25,865
4		1,016,927	21,922	25,282
3		571,510	41,276	23,848
9		440,940	104,833	19,664
11		808,916	25,295	17,288
12		389,791	49,795	18,838
14		599,060	41,485	20,820
10		3,136,014	12,340	24,191

(1 psi = 6,895 N/m^2)

Table B.9. Modulus Values for Lancaster Blvd. Pavement Structure Calculated from FWD Deflection Basin (BISDEF with Core Samples).

Deflection Location	AC (psi)	Untreated Base (psi)	Subgrade (psi)
2	1,652,000	7,856	8,750
3		7,645	9,743
16		6,938	9,116
17		9,317	6,810
5		32,841	8,631
6		32,300	7,843
14		25,630	7,683

(1 psi = 6,895 N/m^2)

Table B.10. Modulus Values for Interstate 5 Pavement Structure Calculated from FWD Deflection Basin (BISDEF with Core Samples).

Slab Number	PCC (psi)	Untreated Base (psi)	Subgrade Modulus (psi)
1	2,869,000	58,957	25,574
11		453,544	10,042
12		172,084	16,076
14		73,792	9,689
10		39,199	19,716
15		136,217	22,656

(1 psi = 6,895 N/m^2)

Table B.11. Modulus Values for Salem Parkway Pavement Structure Calculated from FWD Deflection Basin (BISDEF without Core Samples).

Deflection Location	AC (psi)	CTB (psi)	CTS (psi)	Subgrade (psi)
2	348,469	931,649	75,671	25,865
4	913,111	843,734	27,675	25,282
3	2,989,338	243,487	73,950	23,848
9	364,805	496,724	201,921	19,664
11	836,790	2,109,737	206,570	17,288
12	435,024	685,396	23,071	18,838
14	1,651,514	430,532	47,671	20,820
10	387,573	5,716,788	5,424	24,191

(1 psi = 6,895 N/m^2)

Table B.12. Modulus Values for Lancaster Blvd. Pavement Structure Calculated from FWD Deflection Basin (BISDEF without Cores).

Deflection Location	AC (psi)	Untreated Base (psi)	Subgrade (psi)
2	635,611	11,737	13,539
3	626,676	11,451	14,617
16	807,342	9,545	14,066
17	770,535	12,043	11,660
5	1,867,580	26,047	17,434
6	1,665,458	24,301	16,508
14	1,181,187	22,762	15,574

(1 psi = 6,895 N/m^2)

Table B.13. Modulus Values for Interstate 5 Pavement Structure Calculated from FWD Deflection Basin (BISDEF without Core Samples).

Slab Number	PCC (psi)	Untreated Base (psi)	Subgrade Modulus (psi)
1	2,608,844	48,489	25,644
11	4,329,202	111,444	18,653
12	4,505,473	26,984	21,507
14	3,996,153	24,786	24,286
10	2,666,944	56,947	18,587
15	4,185,419	31,046	24,805

(1 psi = 6,895 N/m^2)

modulus determined from the cores. These results are given in Tables B.14 through B.16. Summary tables and discussion of the results are in Chapter 4.

Table B.14. Modulus Values for Salem Parkway Pavement Structure Calculated from Dynaflect Deflection Basin (BISDEF with AC Core Samples).

Deflection Location	AC (psi)	CTB (psi)	CTS (psi)	Subgrade (psi)
2	863,000	1,247,843	21,882	25,766
4		826,341	33,961	26,687
3		1,264,845	14,177	22,643
9		986,813	12,213	19,664
11		2,106,327	2,160	13,586
12		567,857	16,546	20,195
14		1,046,265	13,835	21,349
10		3,349,232	5,204	21,977

(1 psi = 6,895 N/m^2)

Table B.15. Modulus Values for Lancaster Blvd. Pavement Structure Calculated from Dynaflect Deflection Basin (BISDEF with Core Samples).

Deflection Location	AC (psi)	Untreated Base (psi)	Subgrade Modulus (psi)
2	1,652,000	15,008	8,607
3		13,712	9,338
16		10,466	11,541
17		18,505	7,972
5		27,479	8,362
6		27,583	7,138
14		28,358	7,833

(1 psi = 6,895 N/m^2)

Table B.16. Modulus Values for Interstate 5 Pavement Structure Calculated from Dynaflect Deflection Basin (BISDEF with Core Samples).

Slab Number	PCC (psi)	Untreated Base (psi)	Subgrade Modulus (psi)
1	2,869,000	12,939	16,843
11		41,504	16,359
12		73,844	12,388
14		8,837	18,998
10		135,873	9,184
15		39,359	20,305

(1 psi = 6,895 N/m^2)

APPENDIX C

FORMAT FOR DESIGN MANUAL

This appendix includes a step by step procedure for each overlay procedure. Flowcharts of these procedures are given in Chapter 3. A detailed design manual is presented in Volume III of this report.

I. Asphalt Concrete over Asphalt Concrete

1. Select Analysis Section.
2. Determine predicted traffic for future (y).
3. NDT method 1 - basin (y/n)? (as opposed to NDT method 2 - max. deflection).

   y: Using a backcalculation computer program predict pavement layer moduli for the layers, and determine the structural layer coefficients from figures such as Figure C.1. The layer coefficients are then multiplied by the thickness of their respective layers and summed to obtain the SN_{xeff} of the existing pavement. Go to Step 12.

   n: Go to Step 4.

4. Assume modulus values for layers.
5. Calculated $H_e = .9h_i \sqrt[3]{E_i(1-u_{sg}^2)/E_{SG}/(1-u_i^2)}$

   h_i = thickness layer i (in.)

   E_i = elastic modulus layer i (psi),

   u_i = Poisson's ratio of layer i,

   E_{SG} = subgrade modulus (psi), and

   u_{sg} = Poisson's ratio of subgrade

6. Input a_c = radius of loading plate (in.).

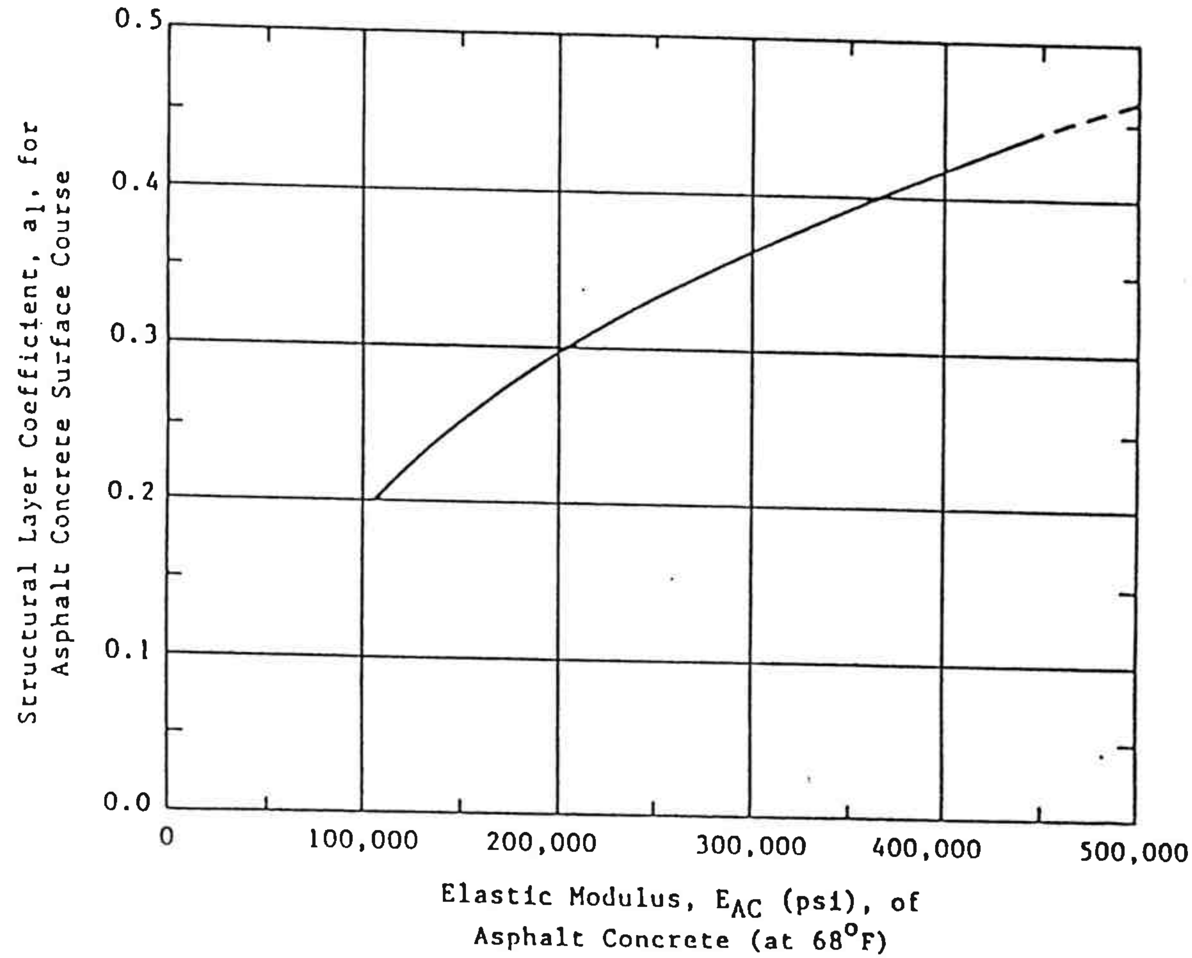


Figure C.1. Chart for Estimating Structural Layer Coefficient of Dense-Graded Asphalt Concrete Based on the Elastic Modulus (2).

7. Find F_b from Figure C.2 knowing H_e/a_c and u_{SG}.

8. Calculate $a_e = a_c/F_b$.

9. Determine S_f from Figure C.3 and solve for E_{SG}

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

where:

P = dynamic load (lbs)

S_f = subgrade modulus prediction factor

d_r = measured NDT deflection at a radial distance

r = radial distance from plate load center

E_{SG} = subgrade modulus of elasticity.

10. Once E_{SG} is calculated, check to ensure $r/a_e > 1$.

11. The structural number is determined using trial and error for the following equation.

Assume SN, Compute F_b, Determine d_o

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

$$\frac{h_e}{a_c} = \frac{209.3\ SN}{a_c}\ \sqrt[3]{\frac{(1-\mu_{sg}^2)}{E_{SG}}}$$

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

When the calculated d_o is approximately the same as the maximum deflection value (adjusted for temp.), then SN_{xeff} can be interpolated.

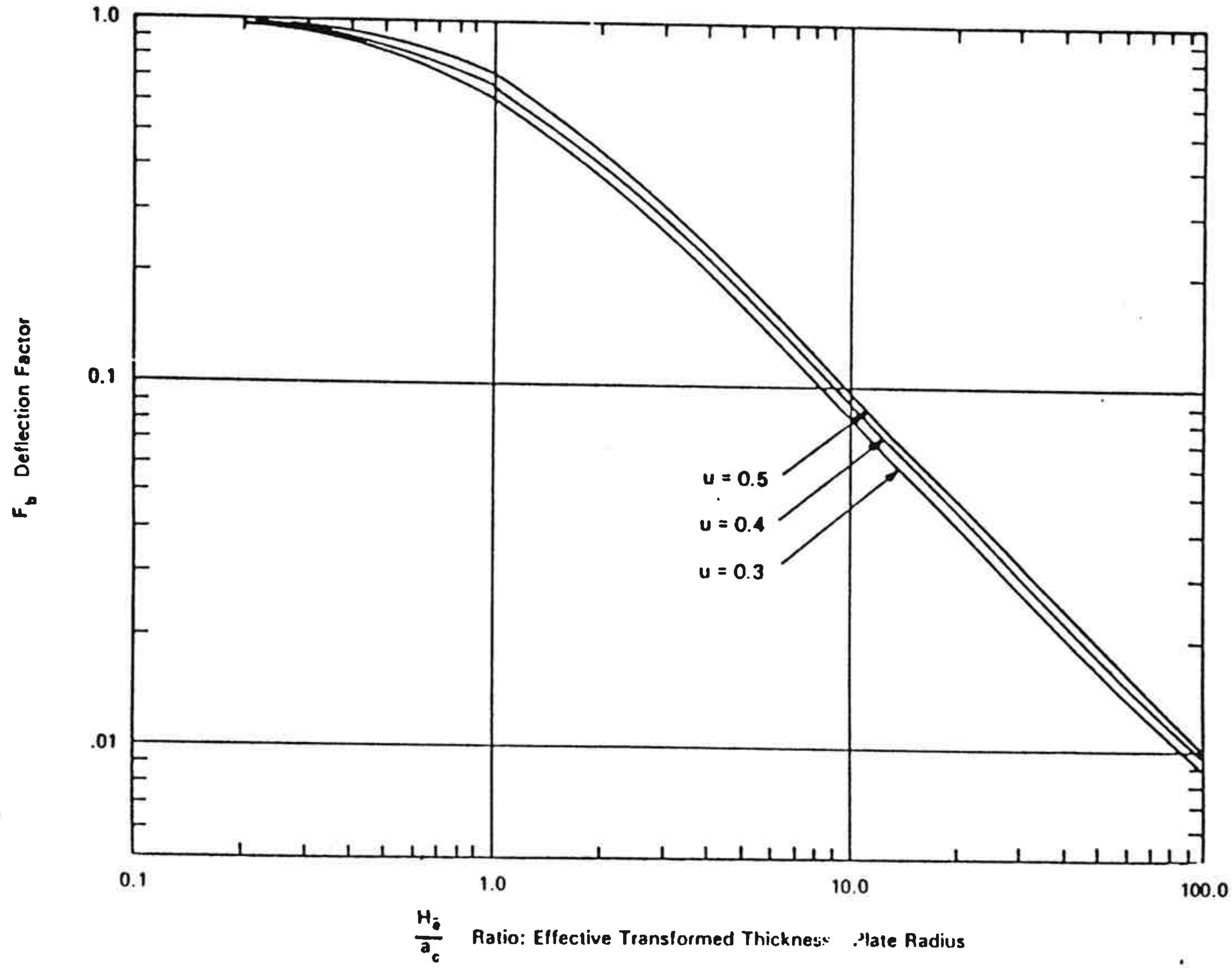


Figure C.2. Deflection Factor vs. Effective Depth Ratio (2).

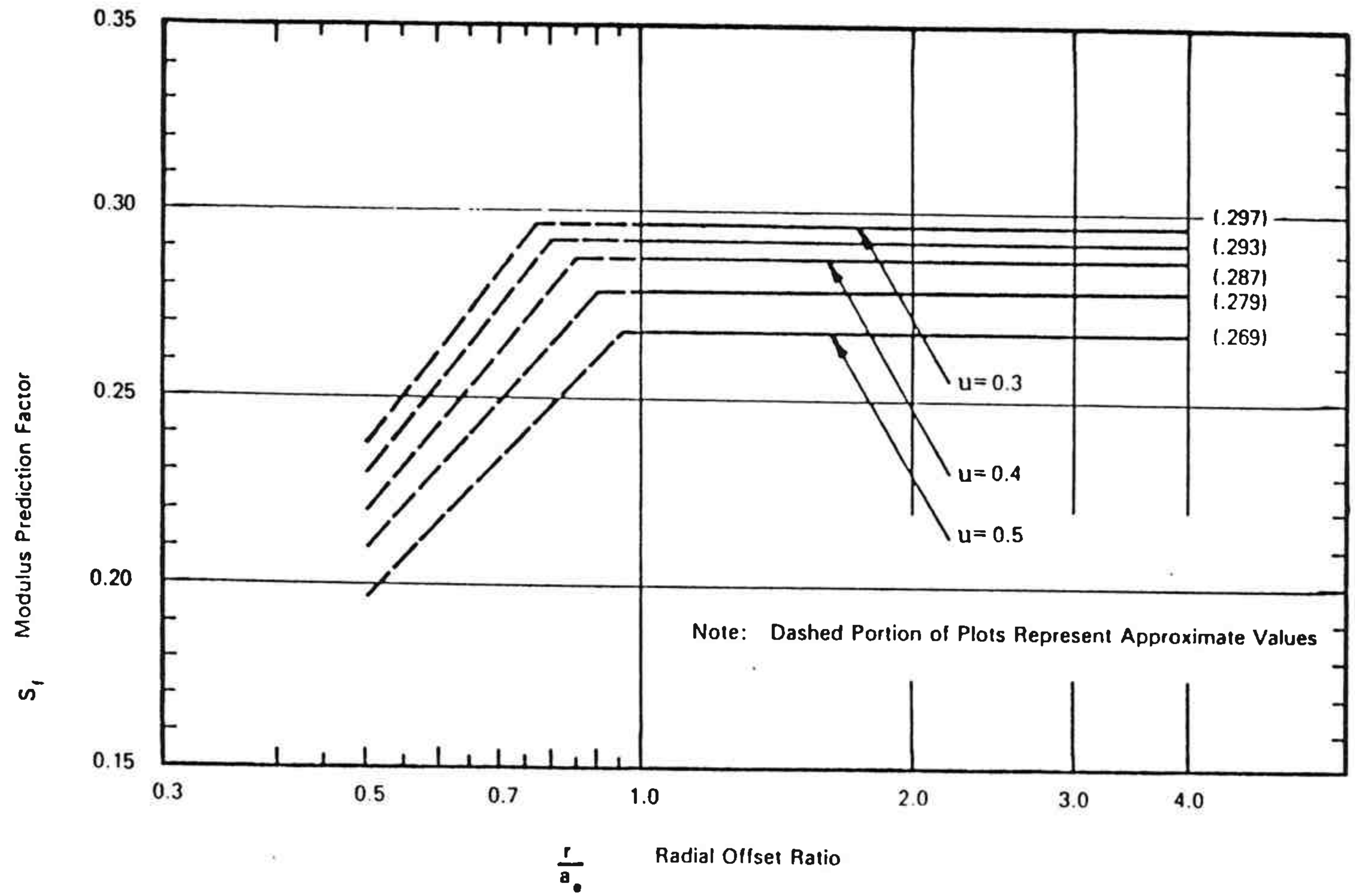


Figure C.3. Subgrade Modulus Prediction Factor vs. Radial Offset Ratio (2).

12. This step determines the structural number, SN_y, as if the pavement was a new design. A reliability level and standard deviation for an appropriate confidence level needs to be assigned. Either the nomograph in Figure 2.10 or the equation below may be used:

$$\log_{10}W_{18} = Z_R S_O + 9.36 \log_{10}(SN + 1) - .20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \log_{10}M_R - 8.07$$

W_{18} = future cumulative traffic from Step 2,

M_R = subgrade modulus from Step (3 or 10),

ΔPSI = difference in desired PSI @ end of service and present P_t,

Z_R = standard normal deviation (Z_R = -1.282 for 90% reliability and -1.645 for 95% reliability), and

S_O = standard deviation (S_O = 0.4 to 0.5 for flexible pavements and 0.3 to 0.4 for rigid pavements).

13. Is historical traffic, x, known (y/n)?

a) y: Historical traffic approach - use Figure 2.10 for original SN of pavement to determine number of repetitions to reach P_t = 2.0 and set equal to N_{fx}.

$R_{LX} = (N_{FX} - X)/N_{Fx}$

Go to Step 15.

n: Want to use NDT approach (y/n)? (This value could be high.)

b) y: NDT approach - knowing initial SN_o and SN_{xeff} from Step (3 or 11) solve for C_x

$C_x = SN_{xeff}/SN_o$

Use C_x in Figure C.4 to find R_{LX}.

Go to Step 15.

n: Use serviceability approach.

Knowing P_t at time overlay and initial SN_o use Figure C.5 to find R_{LX}.

14. Using engineering judgment choose R_{LX} value from one of the above methods.

15. Determine N_{FY} value from SN_y (Step 12) and $P_t = 2.0$ using Figure 2.10.

$$R_{LY} = (N_{FY} - y)/N_{FY}$$

16. Knowing R_{LX} and R_{LY} find F_{RL} from Figure C.6.

17. All values are then substituted into

$$SN_{OL} = SN_y - F_{RL}SN_{xeff}$$

F_{RL} = from Step 16

SN_{xeff} = from Step (3 or 11)

SN_y = from Step 12.

18. For asphalt overlays, the structural layer coefficient may vary from 0.39 to 0.44 (a_i). The required overlay thickness is:

$$h_{OL} = SN_{OL}/a_i$$

where SN_{OL} is determined from Step 17.

II. Asphalt Concrete Over Portland Cement Concrete (Normal Structural Overlay)

1. Determine slab length, thickness PCC (D_o), and thickness of subbase (D_{sb}) from construction records or direct measurements.

2. Determine P_{t1} (at time of overlay).

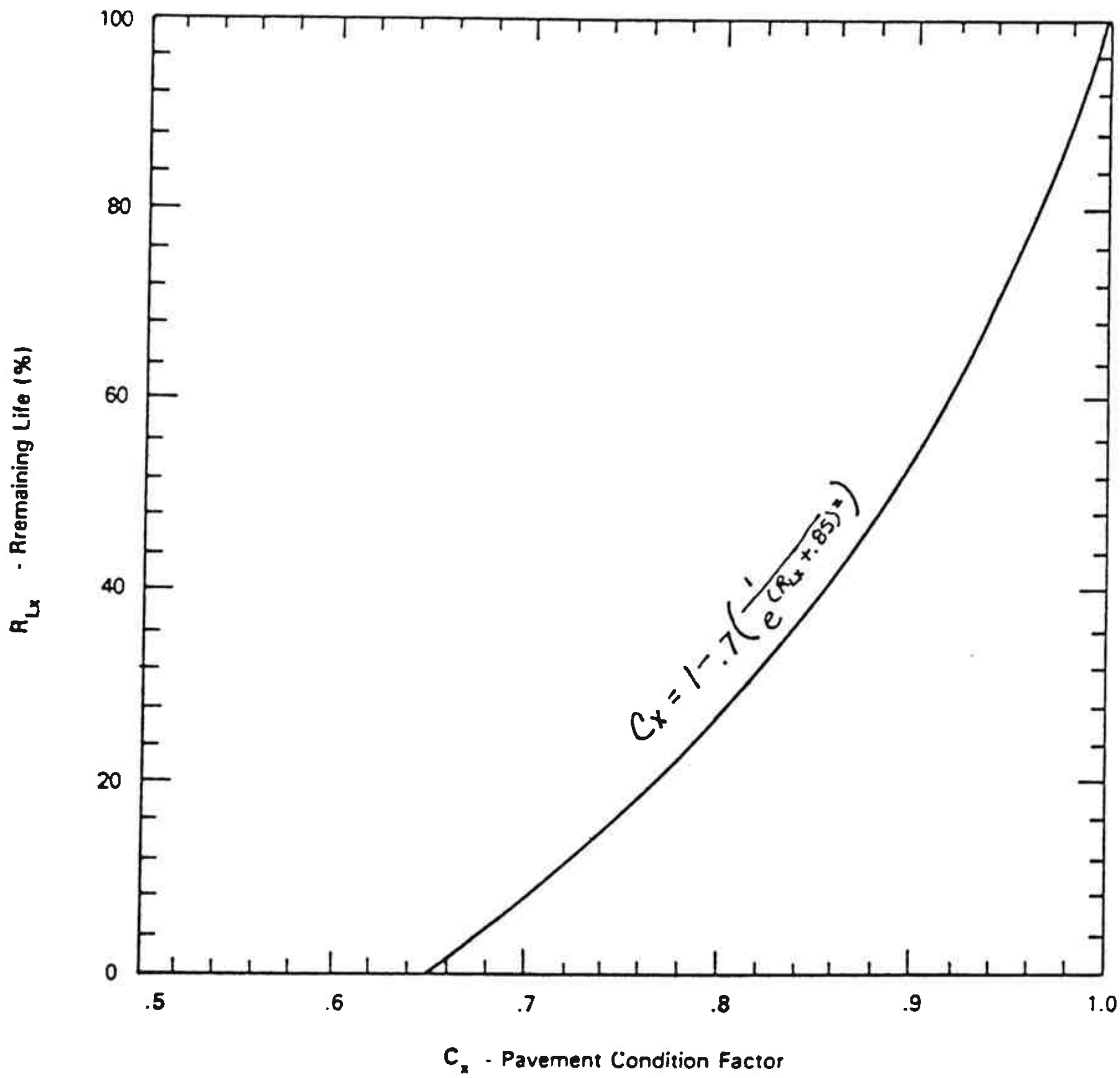


Figure C.4. Remaining Life Estimate Predicted from Pavement Condition Factor (2).

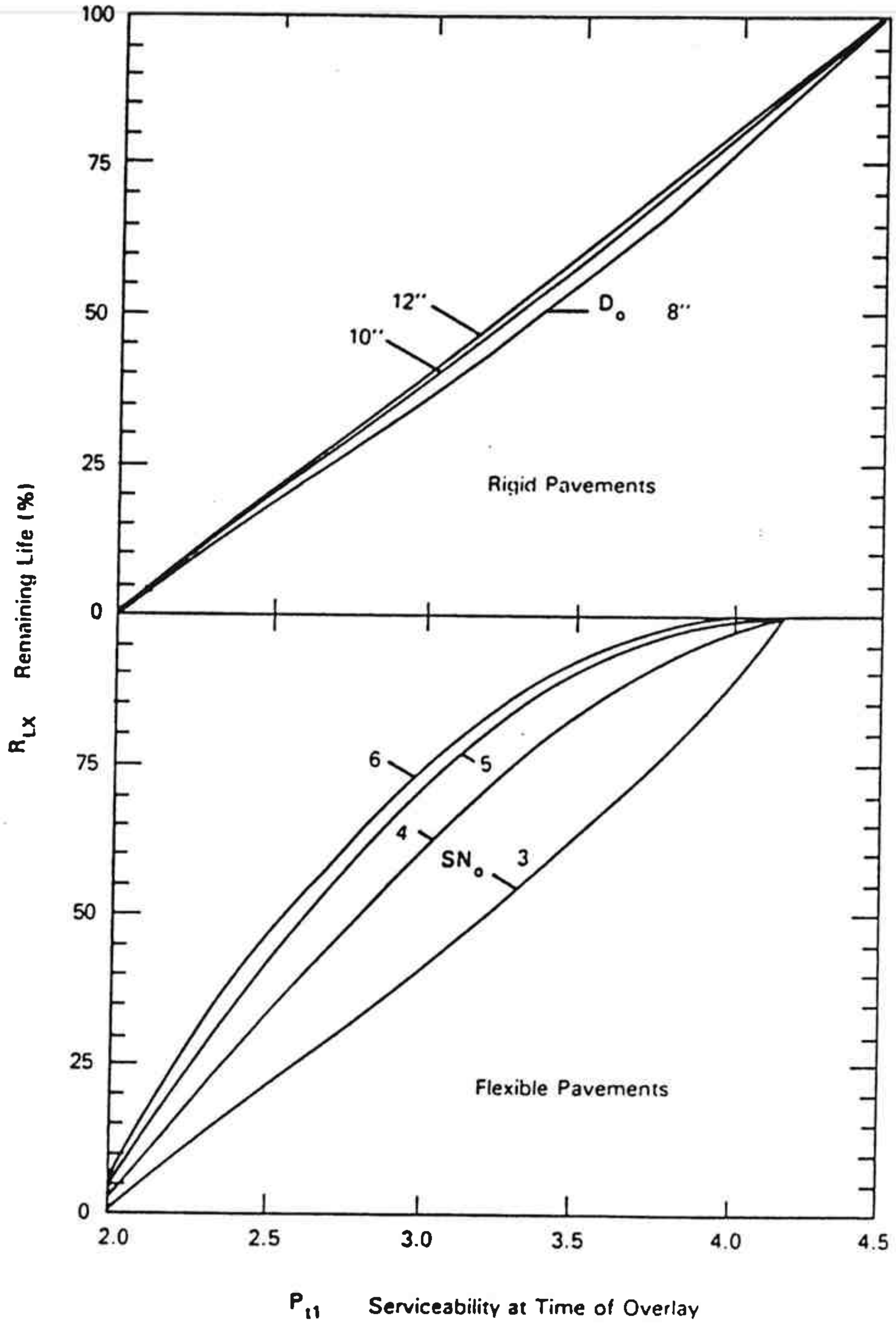
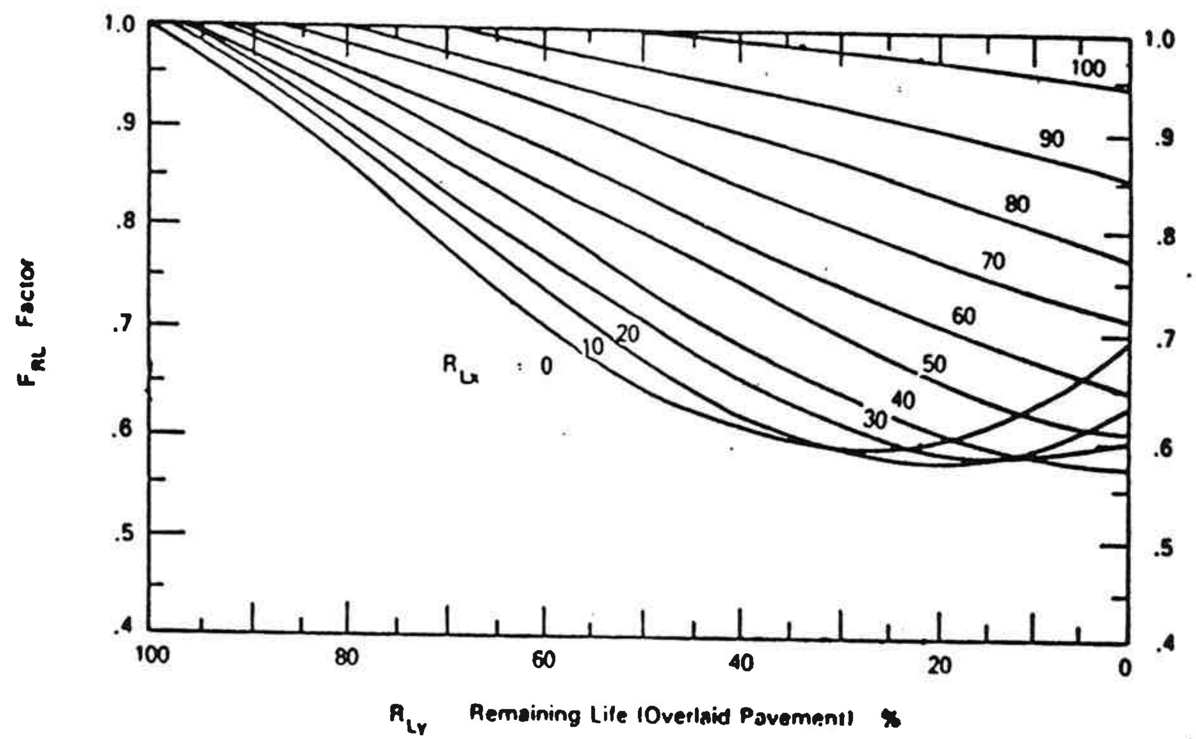


Figure C.5. Remaining Life Estimate Based Upon Present Serviceability Value and Pavement Cross Section (2).



$$F_{RL} = \frac{1-.7\left[\dfrac{1}{e^{(R_{LX}+R_{LY}-.15)^2}}\right]}{\left[1-.7\left(\dfrac{1}{e^{(R_{LY}+.85)^2}}\right)\right]\left[1-.7\left(\dfrac{1}{e^{(R_{LX}+.85)^2}}\right)\right]}$$

Figure C.6. Remaining Life Factor as a Function of Remaining Life of Existing and Overlaid Pavements (2).

3. Get a value for P_{t2} (at end of overlay).

4. Estimate future expected traffic.

5. Calculate SN_{xeff} using NDT Method 1 (y/n)?

   y: Determine $SN_{xeff-rp}$

   $$SN_{xeff-rp} = D_{sb}\ a_{sb}$$

   where:

   a_{sb} = from Figure C.7 and backcalculated E_{sb}

   D_{sb} = Step 1

   There are two alternatives for determining D_{xeff}

   a. Determine E_{PCC} from backcalculation or coring. Knowing E_{PCC} and D_o solve for D_{xeff} from Figure C.8 and use equations from Table 3.9.

   b. Use visual condition rating to find a_{2r} from Figure C.9. Using A_{2x} and D_o input into equation from Table 3.9.

   n: Use NDT Method 2 (as outlined under AC/AC Steps 4-11).

6. Choose a reliability and standard deviation value.

7. SN_y for new design, read off nomograph Figure 2.10.

8. Is previous traffic available (y/n)?

   y: Use Figure 2.10 with original SN_o of pavement and determine number of repetitions to reach P_t = 2.0 to solve for N_{FX}.

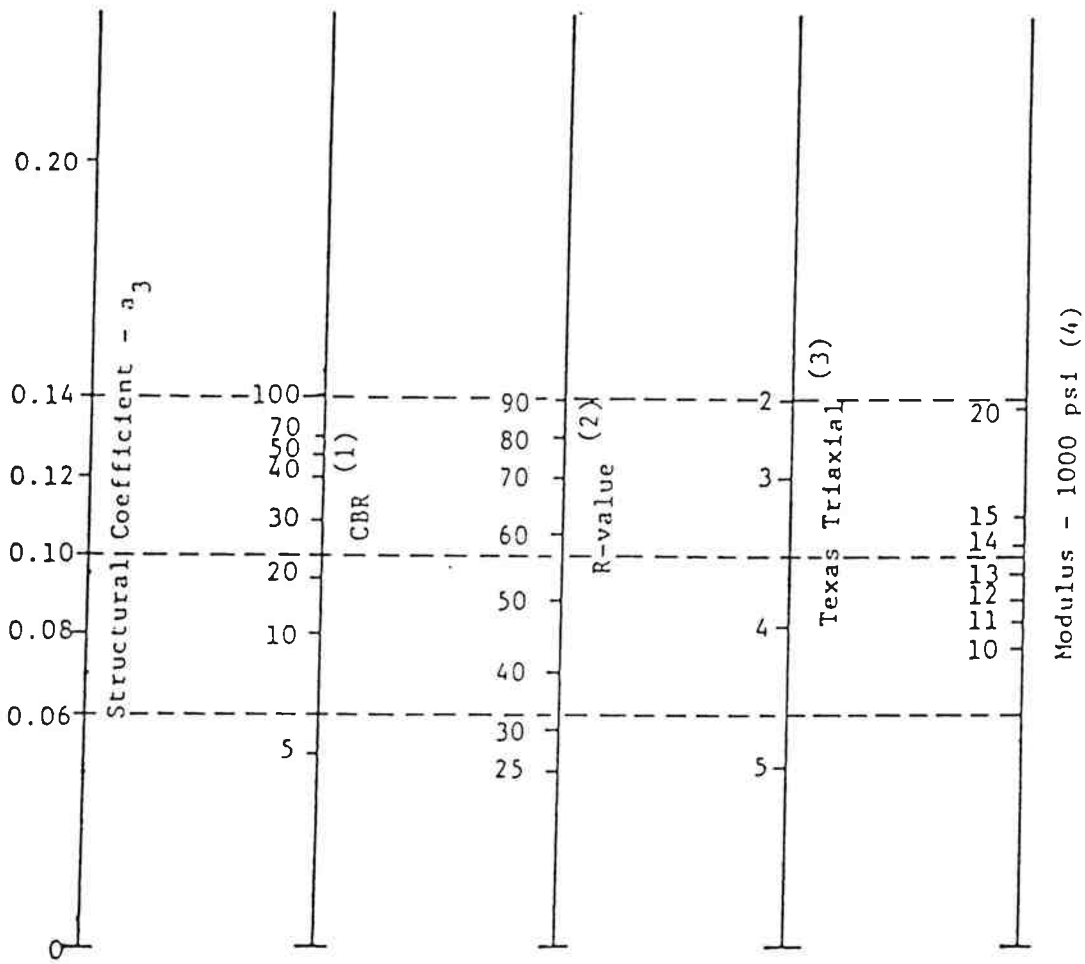
   $$R_{LX} = (N_{FX}-x)/N_{FX}$$

   n: Use NDT results and find C_x (y/n)?

   y: $C_x = D_{xeff}/D_o$

   D_{xeff} = from Step 5a

   D_o = from cores or construction records



(1) Scale derived from correlations from Illinois.
(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.
(3) Scale derived from correlations obtained from Texas.
(4) Scale derived on NCHRP project

Figure C.7. Variation in Granular Subbase Layer Coefficient (A_3) with Various Subbase Strength Parameters (2).

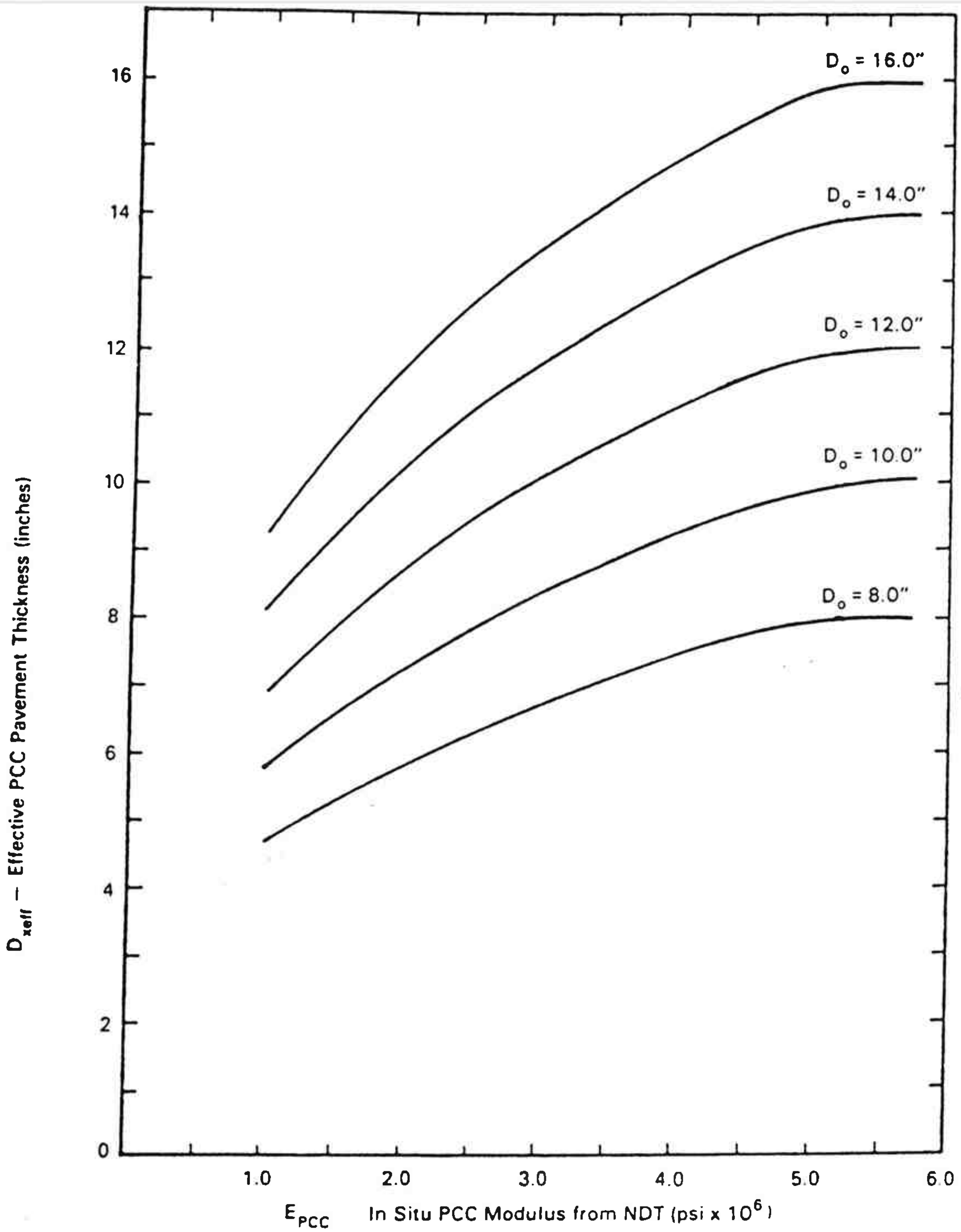


Figure C.8. Determination of Effective PCC Structural Capacity from NDT Derived PCC Modulus (2).

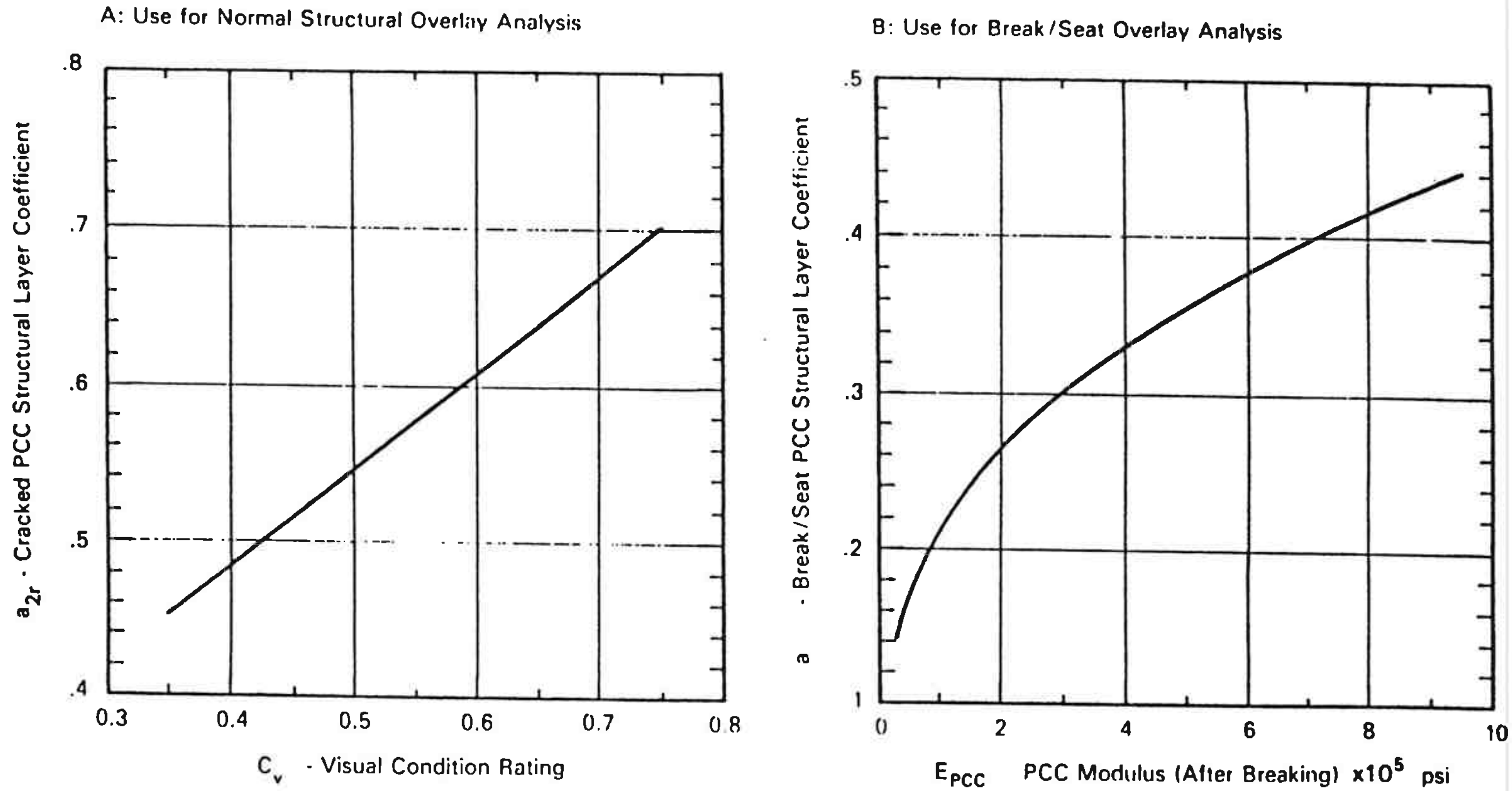


Figure C.9. Structural Layer Coefficients for PCC Layer Used with AC Overlays (Normal Structural and Break/Seat Conditions (2).

Determine R_{Lx} from C_x using Figure C.4.

n: Use serviceability

Knowing D_o (given) and P_{t1} from Step 2 find R_{Lx} from Figure C.5.

9. Find N_{FY} from Figure 2.10 using SN_y from Step 7 and P_{t2} (Step 3) to determine R_{LY} (use y from given).

   $R_{Ly} = (N_{FY}-y)/N_{FY}$

10. F_{RL} factor found using Figure C.6 knowing R_{LX} from Step 8 and R_{Ly} Step 9.

11. Solve for SN_{OL}

    Using Table 3.9

    SN_y = Step 7

    F_{RL} = Step 10

    D_{xeff} = Step 5a or 5b

    $SN_{xeff-rp}$ = Step 5

    D_o = slab thickness

12. Find required thickness

    $$h_{OL} = SN_{OL}/a_i$$

    a_i = value from Figure C.1 knowing E_{AC} for intended overlay (or use .44)

    SN_{oL} = from Step 11.

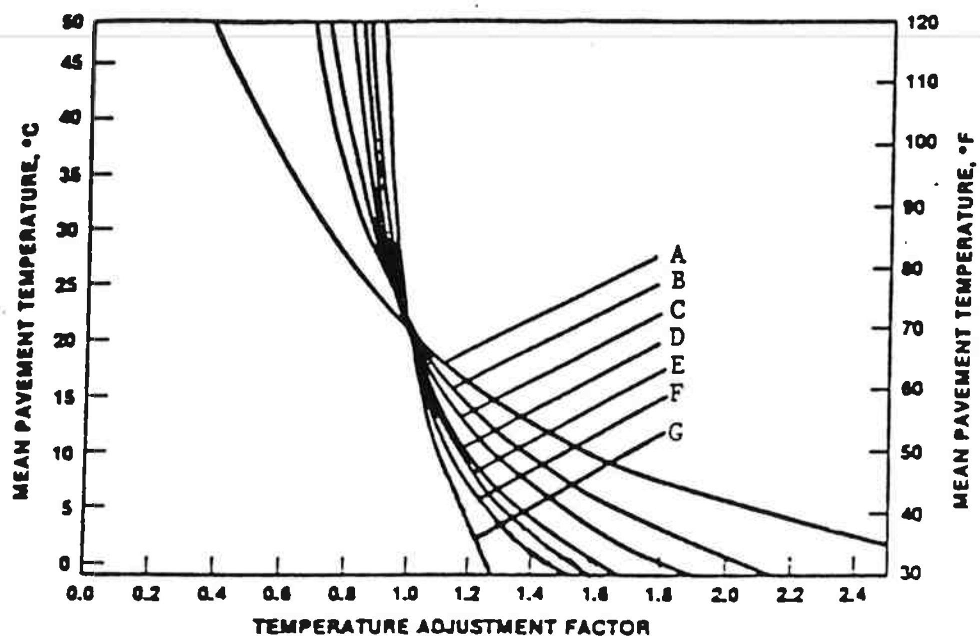
III. Portland Cement Concrete Over Asphalt Concrete

1. Solve for k_d

   $$k_d = F_d \times C_d$$

   k_d = deflection temp adjustment factor to adjust pavement to 100°F

   F_d = adjustment for pavement temperature Figure C.10



Curve Identification

Base Material	Curve (Base Thickness)
Asphalt (Full Depth)	A (All Thicknesses)
Asphalt (Deep Strength)	B (4" of Granular Subbase)*
Portland Cement Concrete	G
Granular (Nonstabilized)	C (6"); D (12"); E (20"); F (25")
Cement-Treated Base	
Sound	D (4"); E (8")
Cracked	C (4"); D (8")

*If more than 4" of granular material present, use "Granular (Nonstabilized) base material category.

Figure C.10. Deflection Temperature Adjustment Factor (2).

C_d = a function of the existing pavement Table C.1

2. Solve for max NDT deflection adjusted to critical in situ asphalt layer temperature

   $d_{oc} = d_o \times k_d$

   d_{oc} = adjusted max deflection

   d_o = unadjusted NDT max deflection obtained at temperature t_p

   k_d = from Step 1

3. Solve for E_c

   $E_c = P/(D_p \times d_{oc})$

   E_c = modulus of composite layers

   P = NDT dynamic load

   D_p = NDT load plate diameter

   d_{oc} = adjusted max deflection

4. Find subgrade reaction k from Figure C.11.

5. Go through procedure for new rigid pavement design using k value determined in Step 4.

6. Use both nomographs (Figures C.12 and C.13) to find D_y

7. D_{OL} = Dy

   D_{OL} = overlay slab required

   Dy = from Step 6

IV. <u>Portland Cement Concrete Over Portland Cement Concrete</u>

1. Decide on bonding option.
2. Determine E_{PCC} from backcalculation or laboratory testing.
3. Knowing E_{PCC} and D_o find D_{xeff} from Figure C.8.
4. Determine D_y from Figures C.12 and C.13.
5. Determine R_{LX} from Step 8 in AC/PCC.

Table C.1. Sample C_d Values for Various Pavement Types (2).

Existing Pavement Type	C_d Value
Full Depth/Deep Strength Asphalt Pavements	1.70
Flexible Pavements with Granular Base/Subbase	1.35
Flexible (Semi-Rigid) Pavements with Cement-Treated Base/Subbase	1.20
Composite Pavement Structures (Asphalt over PCC)	1.05

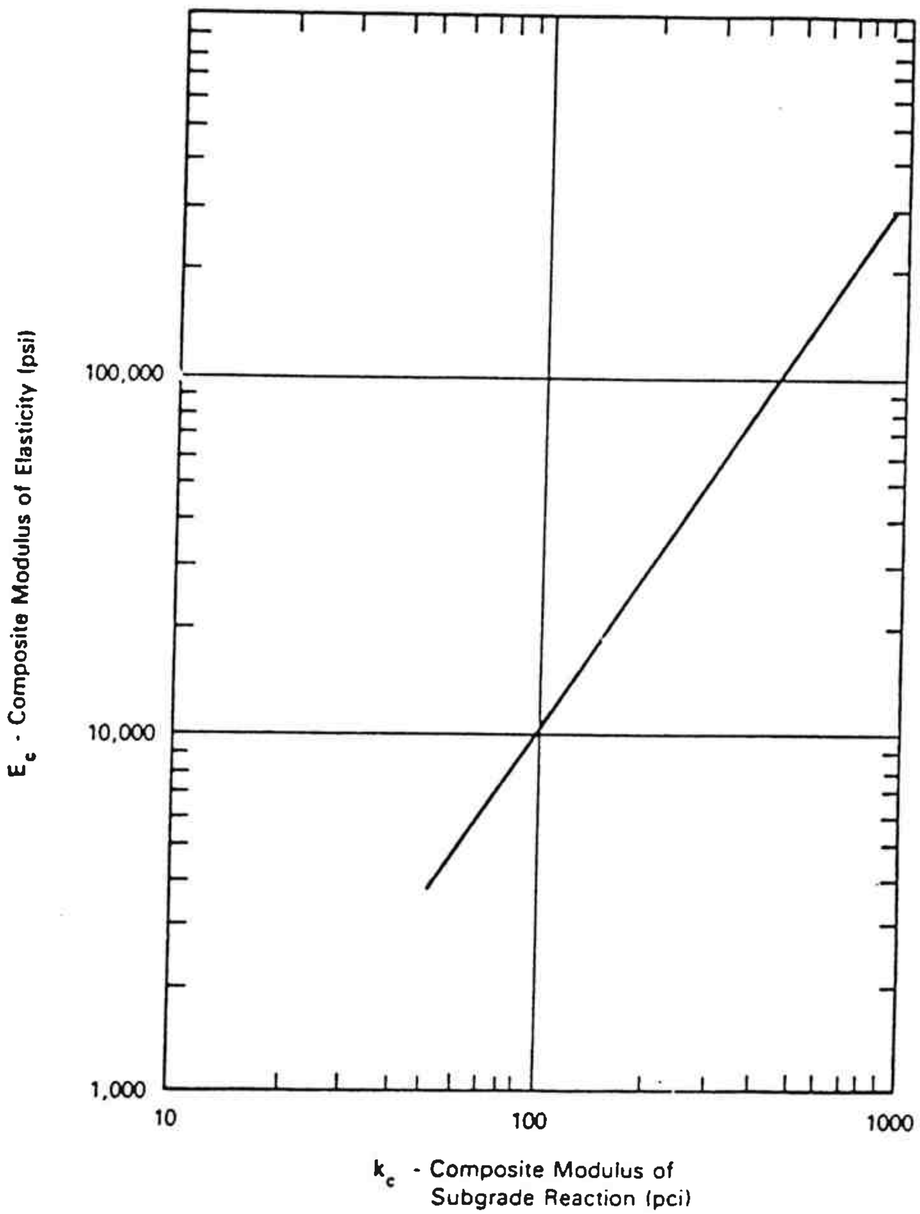
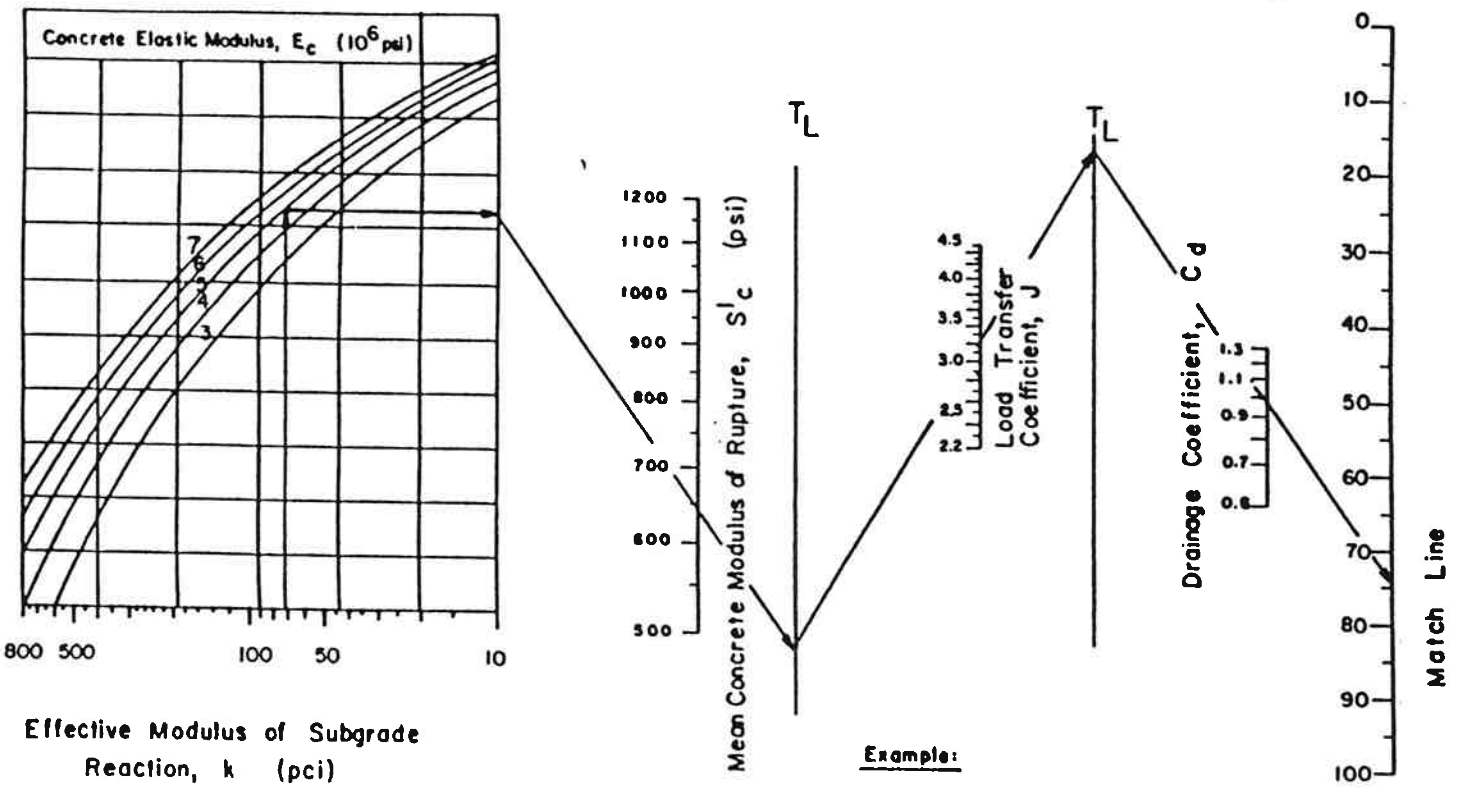


Figure C.11. Relationship Between Composite Modulus of Elasticity and Reaction (2).

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R {*} S_o + 7.35 {*} \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \frac{1.624 {*} 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32 p_t) {*} \log_{10}\left[\frac{S'_c {*} C_d \left[D^{0.75} - 1.132\right]}{215.63 {*} J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$



Example:

k = 72 pci
E_c = 5×10^6 psi
S'_c = 650 psi
J = 3.2
C_d = 1.0

S_o = 0.29
R = 95 % (Z_R = -1.645)
Δ PSI = 4.2 - 2.5 = 1.7
W_{18} = 5.1×10^6 (18 kip ESAL)
Solution: D = 10.0 inches (nearest half-inch, from segment 2)

Figure C.12. Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 1) (2).

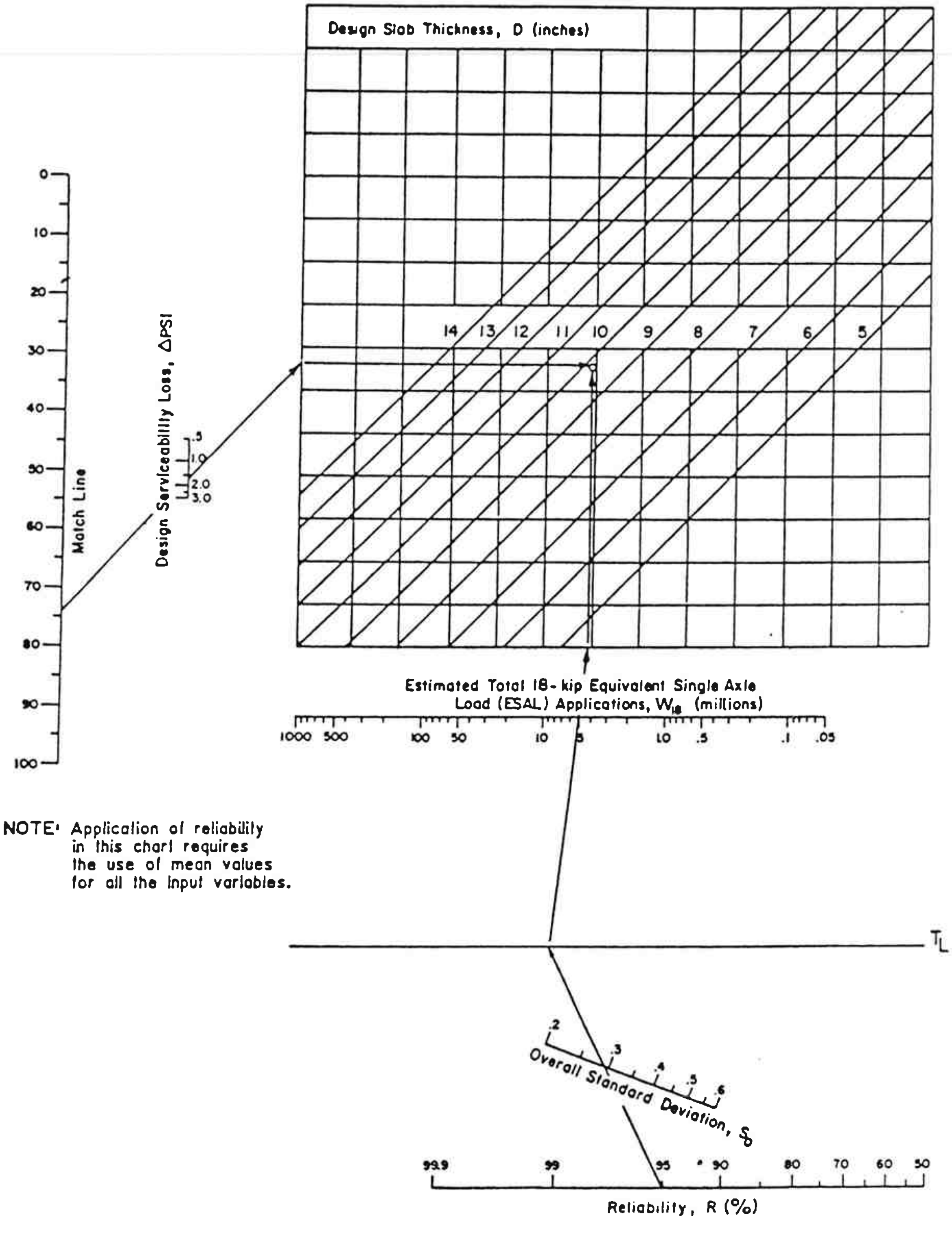


Figure C.13. Design Chart for Rigid Pavements Based on Using Mean Values for Each Input Variable (Segment 2) (2).

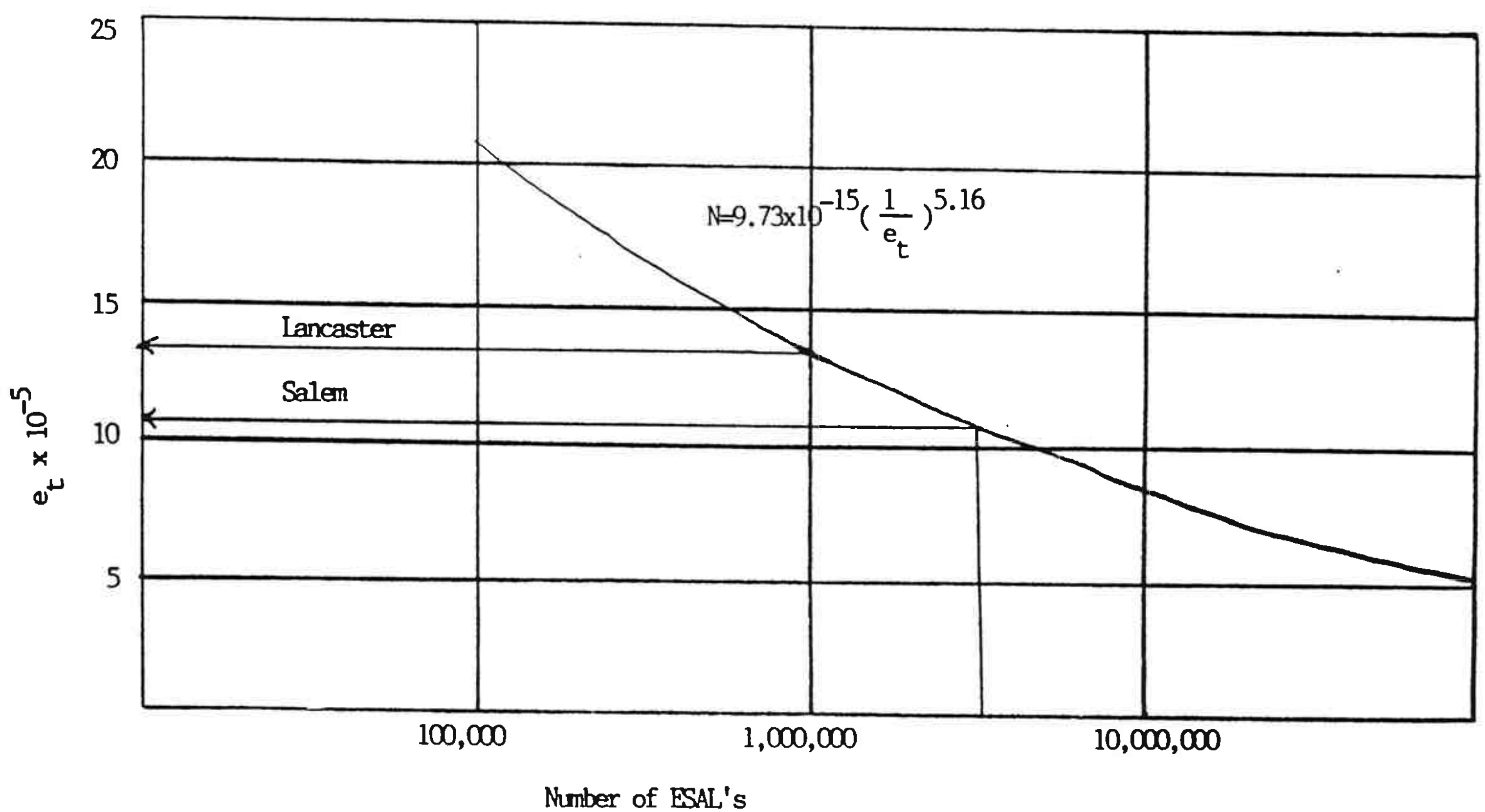


Figure E.1. Fatigue vs. Traffic.

6. Determine R_{LY} from Step 9 in AC/PCC.

7. Find F_{RL} from Figure C.6.

8. Use appropriate equation from Table 2.9.

APPENDIX D

EXAMPLES OF AASHTO METHOD

Example calculations for the AASHTO procedure are contained in this Appendix.

I. Asphalt Concrete Over Asphalt Concrete

A. Salem Parkway

1) Pavement layer thicknesses

3.5 in. AC (asphalt concrete)

10 in. CTB (cement-treated base)

6 in. CTS (cement-treated subgrade)

2) Future traffic in 20 years = 3.2×10^6 ESAL's

3) $P_o = 4.2$, $P_t = 2.5$, $\Delta PSI = 1.7$

4) Calculate SN_o from standard coefficients from AASHTO road test

$$SN_o = (3.5 \text{ in.})(.44) + (10 \text{ in.})(.26) + (6 \text{ in.})(.17)$$
$$= 5.16$$

5) Using NDT method 1 find SN_{xeff}

E_{AC} = 863,000 psi (from core testing)

E_{CTB} = 2,280,000 psi (from core testing)

↓

BISDEF

↓

E_{CTS} = 100,000 psi (lower bound value)

E_{SG} = 14,000 psi

Using layer coefficients from Appendix E

$$SN_{xeff} = (3.5 \text{ in.})(.44) + (10 \text{ in.})(.52) + (6 \text{ in.})(.12)$$
$$= \underline{7.46}$$

6) Determine SN_y from Figure 2.10

Reliability = 95% So = .35 (standard deviation)

Future traffic = 3.2×10^6 ESAL's

E_{SG} = 14,000 psi

$\Delta PSI = 1.7$

From Figure 2.10 $SN_y = 3.5$

7) Remaining Life Factor

a) NDT Approach

$C_x = SN_{xeff}/SN_o = 7.46/5.16 \geq 1.0$

$R_{LX} = 1.0$ from Figure C.4

b) Using Figure 2.10 to find N_{FY} and thus R_{LY}

$SN_y = 3.5$

$\Delta PSI = 2.2$ ($P_o = 4.2$ $P_f = 2.0$)

E_{SG} = 14,000 psi

$\downarrow$

$N_{FY} = 4.9 \times 10^6$ ESAL's (from Figure 2.10)

$$R_{LY} = \frac{N_{FY}-Y}{N_{FY}} = \left(\frac{4.9-3.2}{4.9}\right) = .34$$

Using Figure C.6 ($R_{LX} = 1.0$ and $R_{LY} = .34$)

$F_{RL} = .98$

8) $SN_{OL} = SN_y - F_{RL}(SN_{xeff})$

$SN_{OL} = 3.5 - .98\ (7.46)$

$SN_{OL} < 0$ $\therefore$ no overlay required

B. Lancaster Blvd.

1) Pavement layer thicknesses

3.5 in. AC (asphalt concrete)

18 in. Base (aggregate base)

2) Future traffic in 20 years = 1.0×10^6 ESAL's

3) $P_o = 4.2$, $P_t = 2.5$, $\Delta PSI = 1.7$

4) Calculate SN_o from layer coefficients from AASHTO road test

$$SN_o = (3.5 \text{ in.})(.44) + (18 \text{ in.})(.14)$$
$$= 4.06$$

5) Using NDT Method 2 find SN_{xeff} (from <u>Ullidtz</u> equation rather than AASHTO equations)

a) Equivalent thickness of pavement for AC and base

$$\text{AC:} \quad H_e = .9(3.5) \sqrt[3]{\frac{200{,}000\ (1-.45^2)}{19{,}000\ (1-.35^2)}}$$
$$= 6.7 \text{ in.}$$

assume $E_{AC} = 200{,}000$ psi

$E_{base} = 40{,}000$ psi

$E_{ss} = 19{,}000$ psi

$$\text{Base:} \quad H_e = .9(18) \sqrt[3]{\frac{40{,}000\ (1-.45^2)}{19{,}000\ (1-.35^2)}}$$
$$= 20.1 \text{ in.}$$

$H_e = 6.7 \text{ in.} + 20.1 \text{ in.} = 26.8 \text{ in.} < 36 \text{ in.}$ (maximum h_e for furthest sensor at 36 in.)

b) Ullidtz equation

$$E_{SG}= \frac{(1-u^2)P_o a^2}{rd_r}$$

u = Poisson's ratio

P_o = loading pressure

a = radius loading plate

r = radial distance to furthest sensor

d_r = deflection at furthest sensor

$$E_{SG}= \frac{(1-.45^2)\ \frac{9000}{\pi a^2}\ a^2}{36 \text{ in. } (d_r)}$$

using d_r = 5.08

E_{SG} = 14,000 psi

Substitute E_{SG} in above equation for equivalent thickness to see if H_e changes.

c) h_t = 3.5 in. + 18 in. = 21.5 in.

d) Max deflection = 28.40 x 10^{-3} in from Appendix B (one value)

e) AASHTO regression equations

SN_{xeff} = 3.31

6) Solve SN_y from Figure 2.10

Reliability = 95% Standard Deviation = .35

Traffic = 1.0 x 10^6 ESAL

E_{SG} = 14,000 psi

ΔPSI = 1.7

SN_y = 2.8

7) Remaining Life Factor

a) NDT Approach

$C_x = SN_{xeff}/SN_o = 3.3/4.06 = .81$

$R_{LX} = .32$ from Figure C.4

b) Using Figure 2.10 to find N_{fy}

$SN_y = 2.8$

$\Delta PSI = 2.2$

$E_{SG} = 14,000$ psi

$\downarrow$

$N_{FY} = 1.5 \times 10^6$

$$R_{LY} = \frac{N_{FY}-y}{N_{FY}} = \frac{1.5-1.0}{1.5} = .32$$

c) Using Figure C.6

$F_{RL} = .67$

8) $SN_{OL} = SN_y - F_{RL}\ (SN_{xeff})$

$= 2.8 - .67\ (3.3)$

$= .589$

$h_{OL} = SN_{OL}/a_i$

$h_{OL} = \frac{.589}{.44}$ (for asphalt)

$h_{OL} = 1.34$ in. $\therefore$ use 1.5 in.

II. <u>Asphalt Concrete Over Portland Cement Concrete</u>

A. Interstate 5

1) Pavement layer thicknesses

$D_o = 8$ in. PCC (Portland Cement Concrete)

$D_{sb} = 12$ in. Base (Aggregate Base)

2) Future expected traffic = 20 years (47×10^6 ESAL's)

3) $P_o = 4.2$, $P_t = 2.5$, $\Delta PSI = 1.7$

4) Calculate SN_o from layer coefficients (see Appendix E, Table E.2)

$$SN_o = (8 \text{ in.})(.50) + (12 \text{ in.})(.14)$$
$$= 5.68$$

5) Using NDT method 1 find SN_{xeff}

E_{PCC} = 2,869,000 psi (from core testing)

↓

BISDEF

↓

E_{Base} = 30,000 psi

E_{SG} = 17,000 psi

D_{xeff} = 6.6 from Figure C.7

$SN_{xeff-rp} = D_{Sb}\ a_{Sb} = (.20)(12 \text{ in.}) = 2.4$ in. (a_{Sb} from Appendix E)

$SN_{xeff} = SN_{xeff-rp} + 0.8\ D_{xeff}$ (Table 3.9)

$= 2.4 + 0.8\ (6.6)$

$SN_{xeff} = 7.68$

6) Solve SN_y from Figure 2.10

Reliability = 95% Standard deviation = .35

Traffic = 47×10^6 ESAL

E_{SG} = 17,000

ΔPSI = 2.0

↓

SN_y = 4.7

7) Remaining Life Factor

a) NDT Approach

$C_x = SN_{xeff}/SN_o = 7.68/5.68 > 1.0$

$R_{LX} = 1.0$ from Figure C.4

b) Using Figure 2.10 find N_{FY} and thus R_{LY}

$SN_y = 4.7$

$\Delta PSI = 2.2 \quad (P_o = 4.2,\ P_f = 2.0)$

$E_{SG} = 17{,}000$ psi

$\downarrow$

$N_{FY} = 85 \times 10^6$ from Figure 2.10

$$R_{LY} = \frac{N_{FY}-y}{N_{FY}} = \frac{85-47}{85} = .45$$

Using Figure C.6

$F_{RL} = .99$

8) $SN_{OL} = SN_y - F_{RL}\ (SN_{xeff})$

$= 4.7 - (.99)\ (7.68)$

$SN_{OL} < 0$

$\therefore$ No structural overlay needed

III. <u>Portland Cement Concrete Over Portland Cement Concrete</u>

A. Interstate 5

1) Pavement layer thicknesses

$D_o = 8$ in. PCC

$D_{Sb} = 12$ in. Base

2) Future Expected Traffic in 30 years = 78.95×10^6 ESAL's

3) $P_o = 4.5,\ P_t = 2.5$

$\therefore \Delta PSI = 2.0$

4) Choose bonding option (bonded overlay)

5) Using NDT method 1 find D_{xeff}

E_{PCC} = 2,869,000 psi (from core testing)

↓

BISDEF

↓

E_{Base} = 30,000 psi

E_{SG} = 17,000

D_{xeff} = 6.6 from Figure C.7

6) Solve for k as outlined in AASHTO guide (k = 600 pci)

7) Solve for D_y

reliability = 95% Standard deviation = 3.5

ΔPSI = 2.0

Traffic = 78.95×10^6 ESAL's

Assume:

$C_d = 1.0$

$J = 3.2$ for CRCP

$S'_c = 650$ psi

$E_c = 5 \times 10^6$ psi

↓

$D_y = 13.5$ in.

8) Remaining Life Factor

a) NDT Approach

$C_x = D_{xeff}/D_o = 6.6/8 = .82$

b) Using Figure C.13 find N_{fy} and thus R_{LY}

$$N_{FY} = 100 \times 10^6$$

$$R_{LY} = \frac{N_{FY}-y}{R_{FY}} = \frac{100-78.95}{100} = .21$$

using Figure C.6

$$F_{RL} = .85$$

9) $D_{OL} = D_y - F_{RL}\ (D_{xeff})$

$D_{OL} = 13.5-(.85)(6.6)$

$= 7.89$ use 8 in.

IV. <u>Portland Cement Concrete Over Asphalt Concrete</u>

A. Find k value at 76°F for:

Salem

$k_d = F_d \times C_d$

$C_d = 1.2$ (from Table C.1)

$F_d = .99$ (from Figure C.10)

$\therefore\ k_d = 1.188$

$d_{oc} = d_o\ K_d$

$d_o = 5.22$ (deflection from Appendix B)

$d_{oc} = (5.22)(1.188) = 6.201 \times 10^{-3}$

$E_c = P/(D_p)(d_{oc})$

$P = 9000$ lbs

$D_p = 5.91$ in.

$E_c = 9000/(5.91)(6.201 \times 10^{-3}) = 245{,}600$

$k = 890$ pci from Figure C.11 (design value) so put directly into equation.

<u>D_y = 7.5 in.</u> from Figures C.12 and C.13

B. Find k value at 108°F for

Lancaster

$k_d = F_d \times C_d$

$C_d = 1.35$ (from Table C.1)

$F_d = .90$ (from Figure C.10)

$\therefore k_d = 1.215$

$d_o = 24.80$ (deflections x 10^{-3} from Appendix B)

$d_{oc} = (1.215)(24.80) = 30.13$

$E_c = P/(D_p)(d_{oc}) \qquad d_{oc} = d_o\ K_d$

$P = 9000$ lbs

$D_p = 5.91$ in.= radius

$E_c = 9000/(5.91)(30.13 \times 10^{-3}) = 50{,}542$

$k = 280$ pci from Figure C.11

$D_y = 7$ in. from Figure C.12 and C.13

APPENDIX E

EXAMPLES OF OTHER PROCEDURES

Examples of calculation for other overlay design procedures and assigned layer coefficient values are contained in this appendix.

See Figures E.1 through E.4 for relationships between traffic, strains, and required thicknesses.

Table E.1. Calculations for Other Overlay Design Procedures.

Caltrans (9)

Project	$\bar{x}$	s	δ_{80}	Traffic ($x10^6$ ESAL's)	δ_t	% Reduction	t_{AC} (in.)
Salem (FWD)	4.87	1.16	5.84	3.2	15	0	0
Lancaster (FWD)	23.75	4.36	27.41	1.0	18	34.3	2.0

Asphalt Institute (6)

Project	$\bar{x}$	s	f	c	Traffic ($x10^6$ ESAL's)	δ_{rrd}	t_{AC} (in.)
Salem (FWD)	4.87	1.16	.98	1.2	3.2	8.46	0
Lancaster (FWD)	23.75	4.36	.80	1.2	1.0	31.17	0

ODOT (31)

Project	$\bar{x}$	s	Temp. °F	δ_{80}	Traffic ($x10^6$ ESAL's)	δ_t	% Reduction	t_{AC} (in.)
Salem (Dynaflect Converted)	5.73	1.91	.96	7.11	3.2	15	0	0
Lancaster (Dynaflect Converted)	32.08	5.74	.81	30.81	1.0	18	41.5	2.6

FHWA-ARE (14)

for h_o = 0 in.

Project	ϵ_t ($x10^4$)	ϵ_c ($x10^{-4}$)	Traffic ($x10^6$ ESAL's)	Fatigue t_{AC} (in.)	Rutting t_{AC} (in.)
Salem	.2233	- .7997	3.2	0	0
Lancaster	1.579	-2.713	1.0	1.0	0

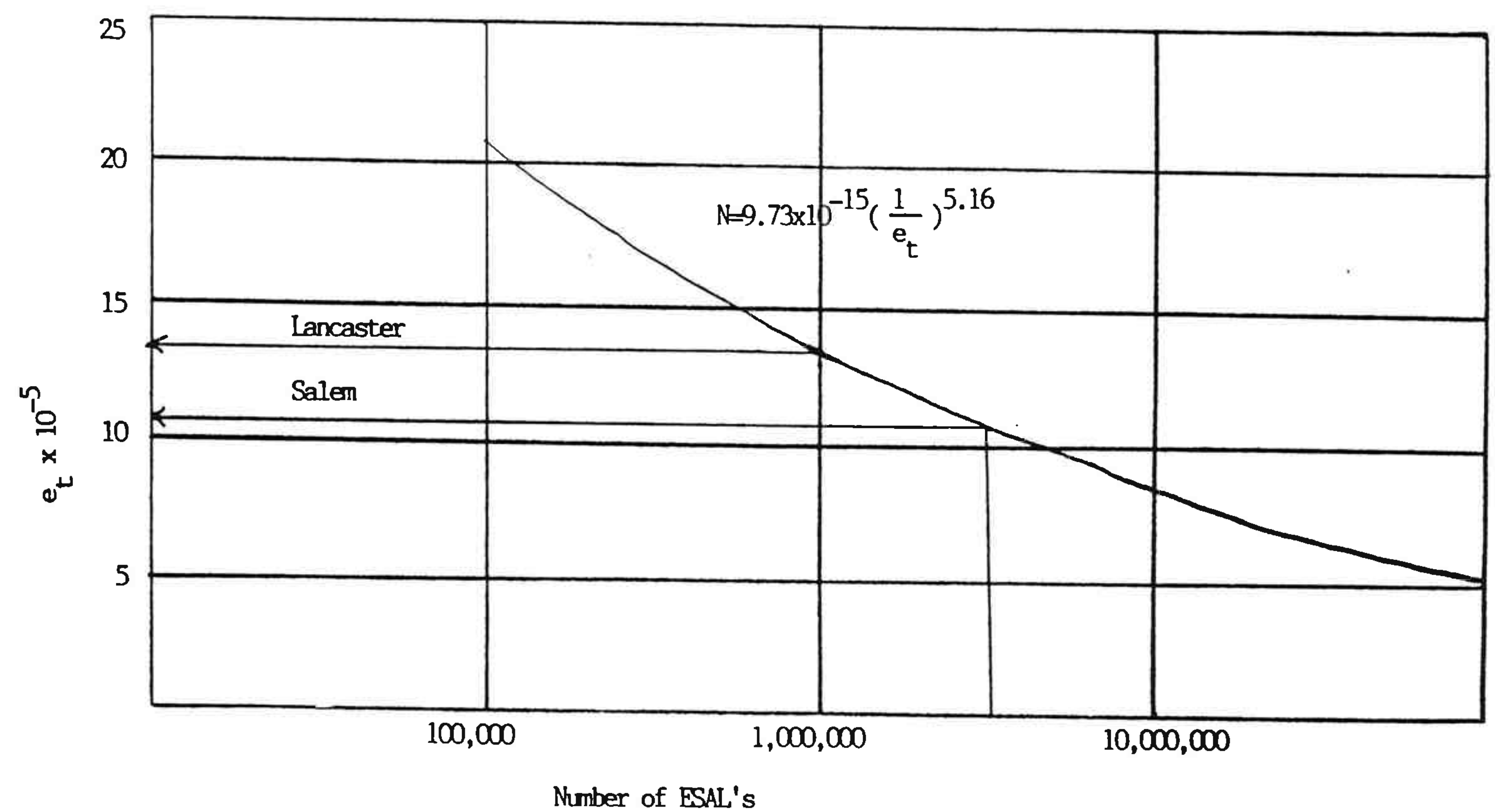


Figure E.1. Fatigue vs. Traffic.

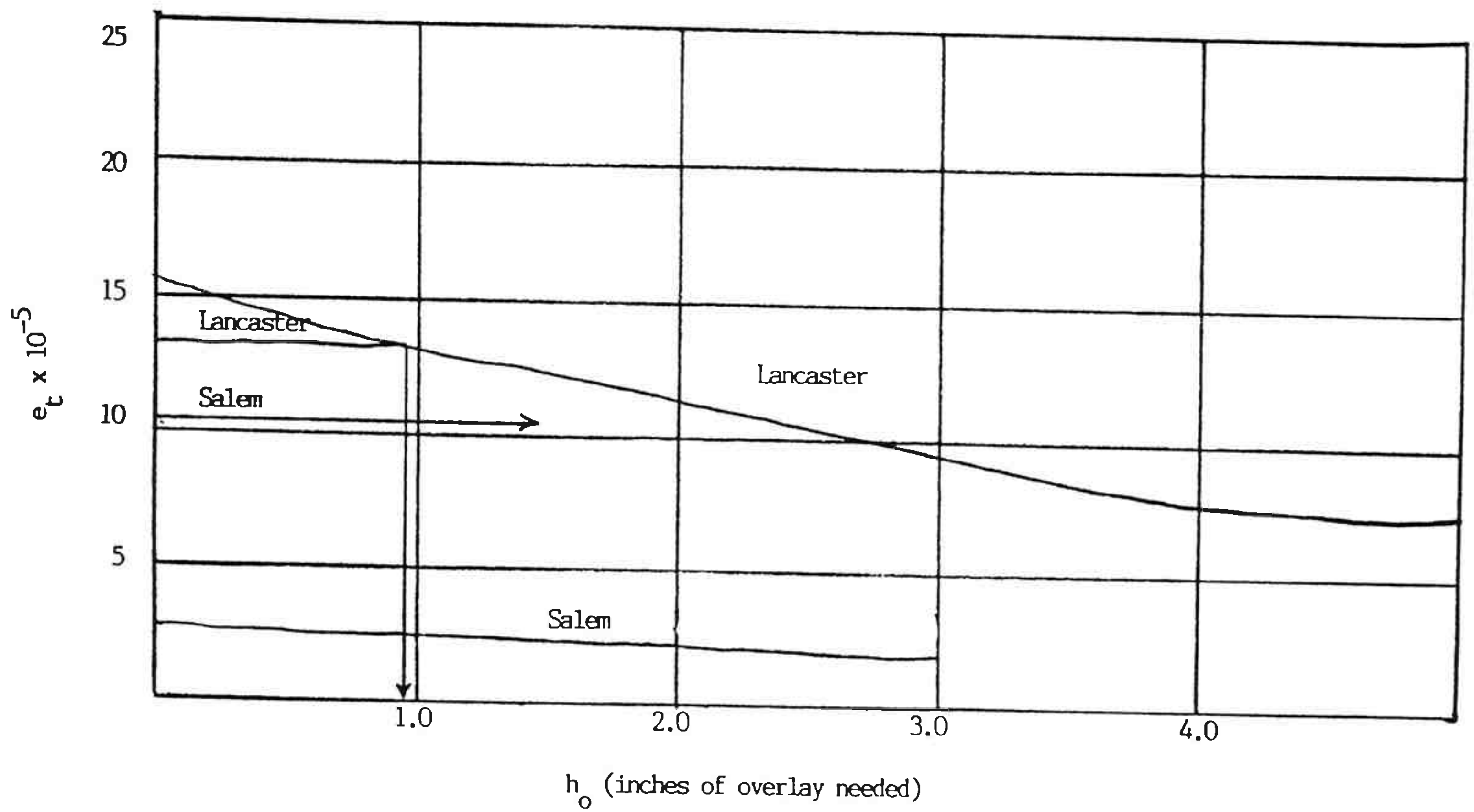


Figure E.2. Thickness vs. Strain (Fatigue).

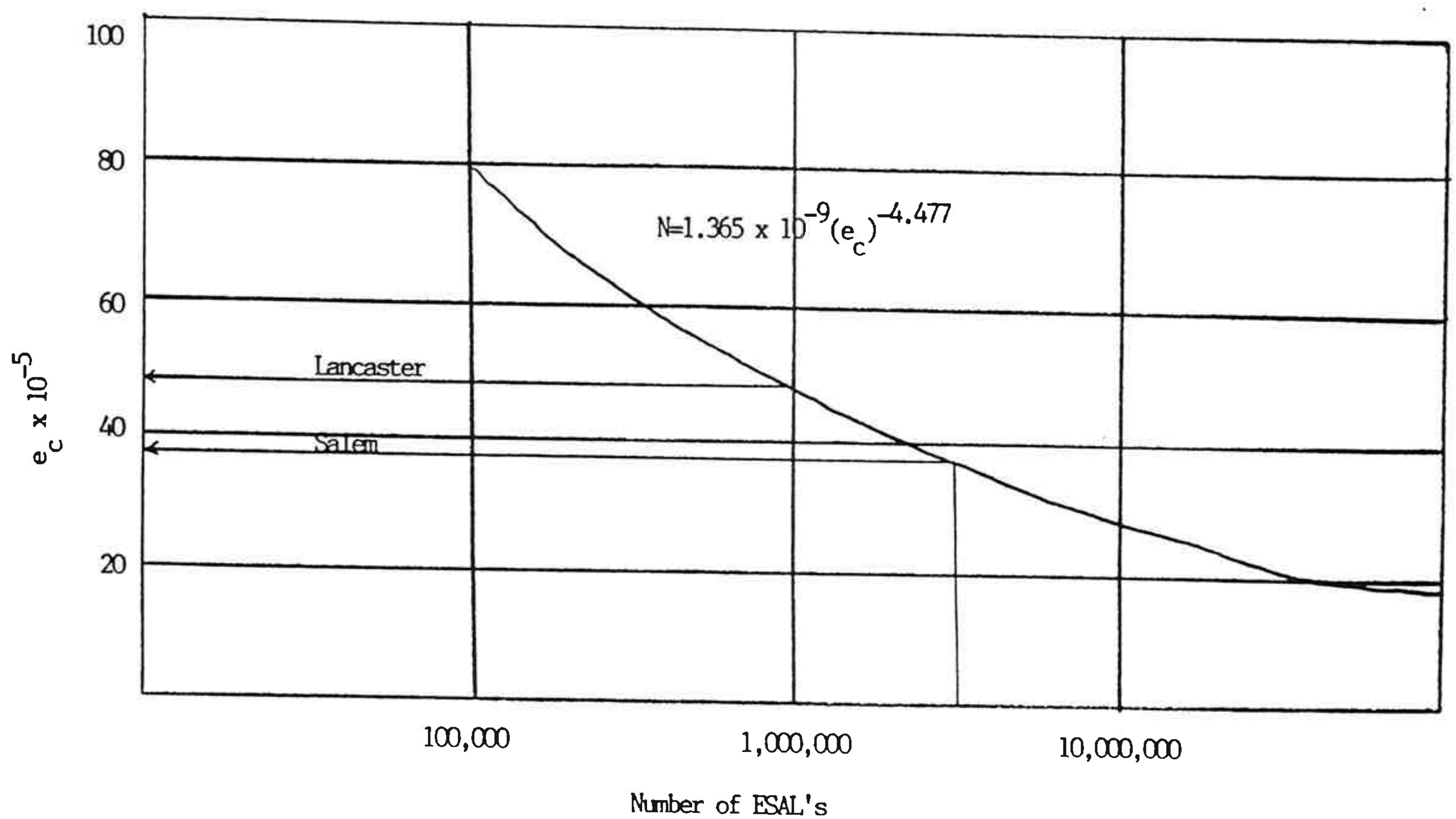


Figure E.3. Rutting vs. Traffic

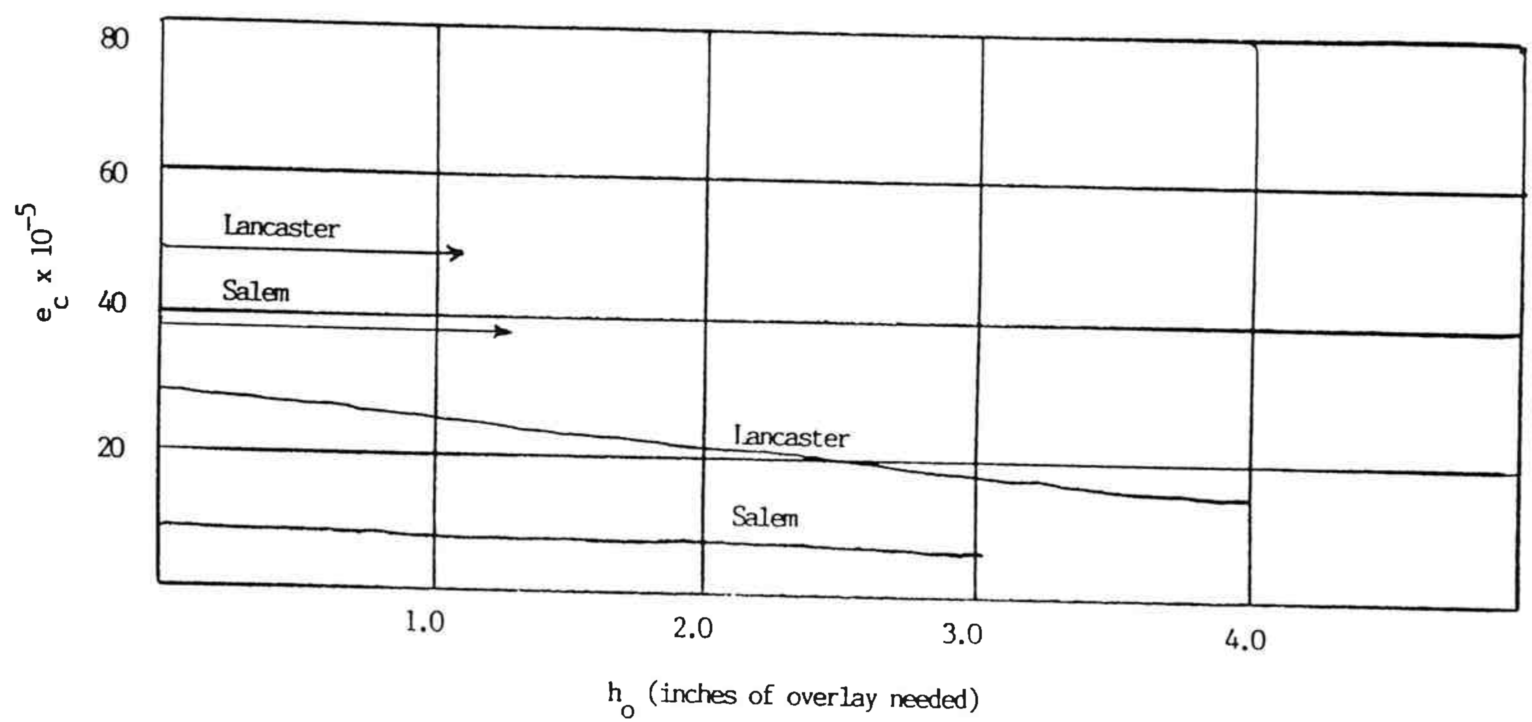


Figure E.4. Thickness vs. Strain (Rutting).

Table E.2. Typical Layer Coefficients Assignments from AASHTO Charts (2)

a) Asphalt Concrete Surface

AC (psi)	a_i	Source
863,000*	.44	(standard)
1,652,000*	.44	(standard)
700,000	.44	(standard)

b) Cement-Treated Base

CTB (psi)	a_i	Source
2,280,000*	.52	(estimated)
500,000	.115	(CTB chart)
598,000	.155	(CTB chart)

c) Cement-Treated Subgrade

CTS (psi)	a_i	Source
100,000*	.12	(bituminous-treated (only one with 100,000 psi))
300,000*	.28	(bituminous-treated)
840,000	.23	(CTB chart)
350,000	.075	(CTB chart)
1,000,000	.265	(CTB chart)
770,000	.21	(CTB chart)

d) Aggregate Base

Base (psi) - Lancaster	a_i	Source
15,600	.068	(granular base)
13,000	.055	(granular base)
46,300	.185	(granular base)
35,400	.16	(granular base)
87,856*	.04	(granular base)
9,317*	.04	(granular base)
32,481*	.14	(granular base)

e) Bituminous-Treated Base

Base (psi) - Interstate 5	a_i	Source
87,000	.10	(bituminous)
200,000	.22	(bituminous)
123,000	.14	(bituminous)
30,000	.03	(bituminous)
5,700*	.04	(bituminous-granular base)
29,056*	.10	(bituminous-granular base)
126,782*	.17	(bituminous)
150,000*	.18	(bituminous)