PAVEMENT RESEARCH
at the
WASHINGTON STATE UNIVERSITY
TEST TRACK

VOLUME FIVE

EVALUATION AND ANALYSIS OF RESULTS

FROM EXPERIMENTAL RINGS NO. 1 - 4

Report to the Washington State Highway Commission
Department of Highways

on Supplement to Research Project Y-993

by

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ABSTRACT

The purpose of this study was threefold: (1) Develop empirical equivalencies from all four rings, (2) develop a design method for overlays based on field deflections; and (3) evaluate the validity of data obtained from instrumentations in terms of n-layer elastic theory and then develop theoretical equivalencies.

This was done. Field equivalencies were developed and they indicate the superiority of the treated base materials over the untreated. A design method was developed which could be used for predicting when an overlay was needed and what thickness was needed to withstand certain equivalent wheel loads and deflections.

Using computer programs for n-layer elastic theory developed by Chevron Research Company, deflection stresses and strains were computed and compared with field data. Assumptions about the material behavior and condition were made based on laboratory data obtained from The Asphalt Institute and field knowledge, and were used to help predict the behavior of pavements. The results were encouraging and indicate that field measurements generally were comparable with elastic layer theory predictions. This will help to develop and modify existing design limits for stresses, strains and deflection for future work.

Equivalencies based on theoretical deflections, stresses and strains indicate the difficulty of assigning precise values. These values also indicate the superiority of treated materials over the untreated materials.

INTRODUCTION

The Washington State University Test Track, officially called the G. A. Riedesel Pavement Research Facility[1], has been in operation since February 1965. During that period, a total of four experimental rings have been built and tested to various degrees of failure. The purpose for building and testing these rings was to study bases composed of different materials of varying thicknesses in order to evaluate their strengths and determine their relative equivalencies. Experimental Rings #1 and #2 were part of contract Y-651, while Rings #3 and #4 were under contract Y-993.

The four experimental rings have been completed and tested. Experimental Ring #1, consisting of 6 sections composed of cement-treated, Class "E" asphalt concrete, and screened aggregate asphalt concrete bases covered with a uniform depth of Class "B" asphalt concrete pavement, was tested from February 1965 to May 1966. The description and results of Ring #1 were reported in the Highway Research Section Publication H-28 (1)[2].

Experimental Rings #2, #3, and #4 were similar in that each ring had 12 experimental sections and three different base materials; each base material consisted of 4 sections of varying base thickness. The untreated crushed rock base material was used in the control sections for all three rings. The pavement consisted of a uniform depth of Class "A" asphalt concrete in all rings. Rings #2 and #3 were similar in that untreated, emulsion treated, and

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[1]Named in honor of G. A. Riedesel, retired head of the Highway Research Section, and the principal originator and designer of the Washington State University Test Track.

[2]Figures in parentheses refer to specific reports in the References.

screened special aggregate asphalt concrete bases were tested, while in Ring #4, sand-asphalt, Class "F" asphalt concrete bases and untreated bases were tested. Rings #2, #3, and #4 were tested during October 1966 to May 1967, September 1967 to July 1968, and November 1968 to August 1969, respectively. All rings (1-4) were temporarily shut down during the winter months of December to March. The description and results of Rings #2, #3, and #4 were reported in Highway Research Section Publications H-29 (2), H-30 (3), and H-31 (4), respectively.

During this 5-year period much data from instrumentation were obtained. Although some of it has been evaluated and analyzed, a more complete analysis emcompassing all the treating periods still remains to be done. A supplement to contract Y-993 with the Washington Highway Department was agreed upon for the purpose of analyzing and evaluating this data with respect to strains, stresses, and deflections, both observed and predicted, and on a theoretical and empirical basis. This report covers this aspect of the project.

These projects were conceived and initiated by the Highway Research Section, College of Engineering Research Division, Washington State University. Financing was a joint undertaking among the University, the Washington Highway Department, the Bureau of Public Roads of the Federal Highway Administration, the Department of Transportation, as an HPR federal aid research project, and The Asphalt Institute, which provided professional guidance in design planning and in evaluation of projects. The Asphalt Institute participated in Rings #2, #3, and #4.

EXPERIMENTAL PAVEMENTS

Full description of experimental Rings #1, #2, #3 and #4 have been provided in previous reports (1, 2, 3 and 4), and in monthly and quarterly progress reports (5, 6, 7). Figure 1 shows the arrangement of the circular test track for Ring #1, the centerline of which is 83 feet in diameter. Six sections of 43.25 feet in length were constructed 8 feet wide with no transition zones. Experimental Rings #2, #3 and #4 had twelve sections 18 feet in length which were constructed 8 feet wide with transition zones approximately 4 feet between adjoining sections. Figure 2 shows the arrangement of the sections for Rings #2, #3 and #4. Shoulders four feet wide on each side completed the plan arrangement for all four rings.

Typical cross-sections of the pavement structure are shown in Figures 3 and 4; the former for Ring #1, while the latter figure shows the cross-section for Rings #2-#4. The thickness and materials of each test section are listed in Tables 1, 2, 3 and 4 for Rings #1, #2, #3 and #4, respectively. As indicated, the surface course and the subgrade were uniform for all sections with the base type and thickness being the principal variable.

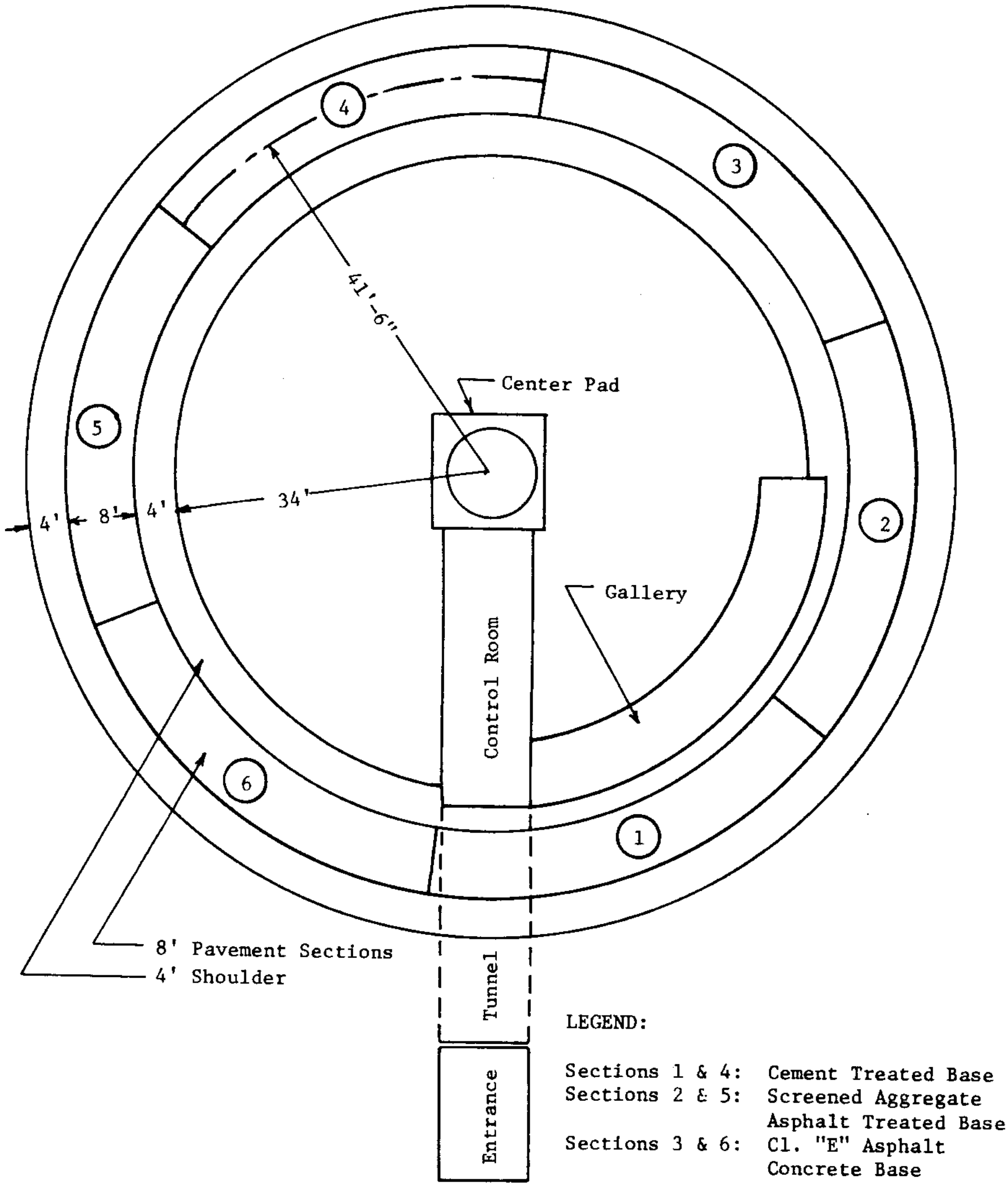
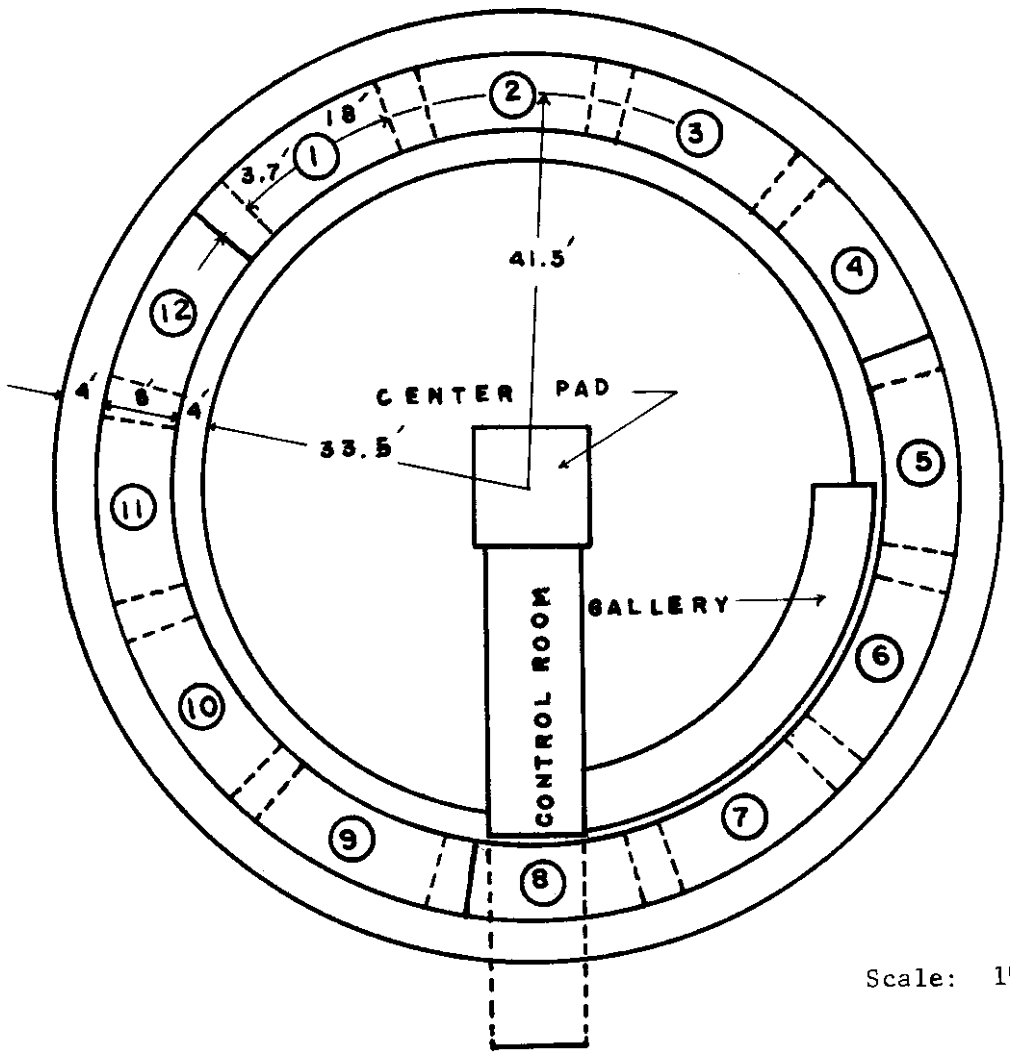


FIGURE 1
PLAN VIEW OF PERMANENT STRUCTURE AND THE PAVEMENT SECTIONS
RING #1

PLAN VIEW

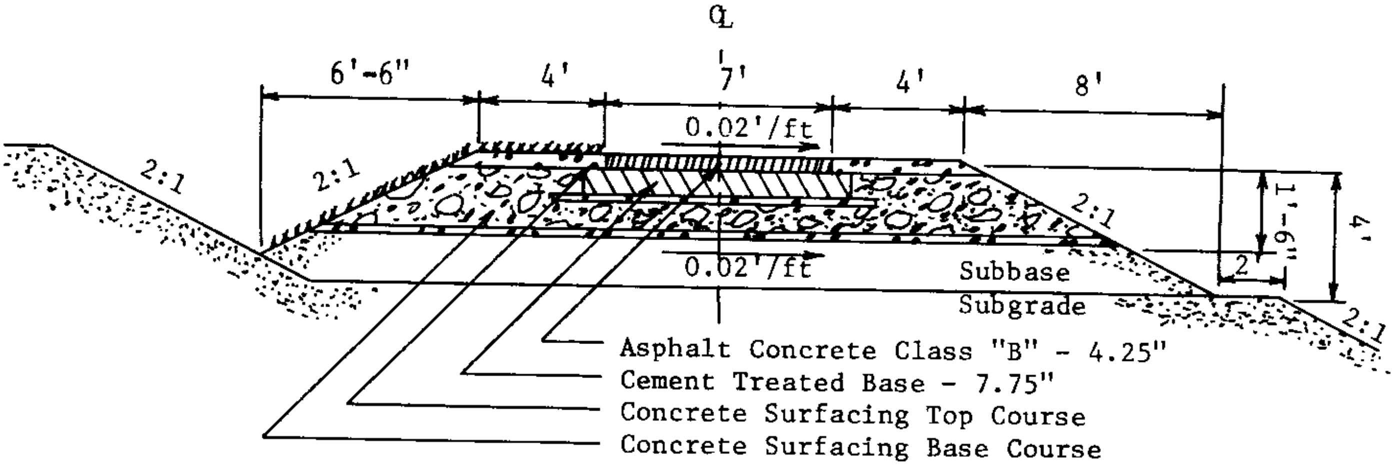


Scale: 1" = 20'

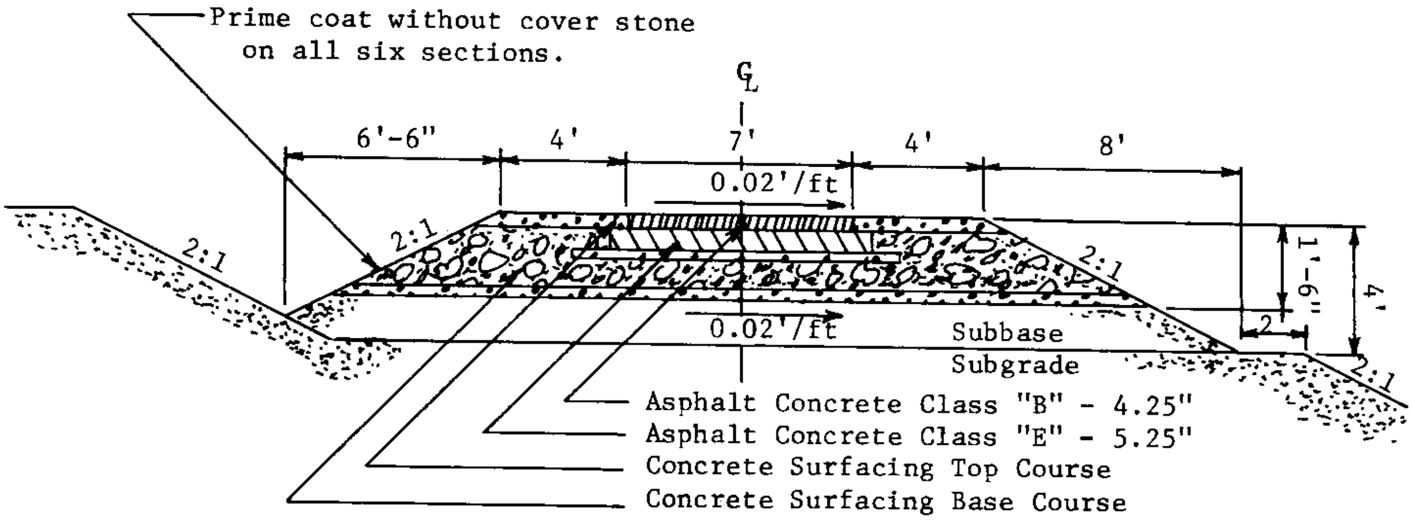
FIGURE 2

PERMANENT STRUCTURES AND PAVEMENT SECTIONS

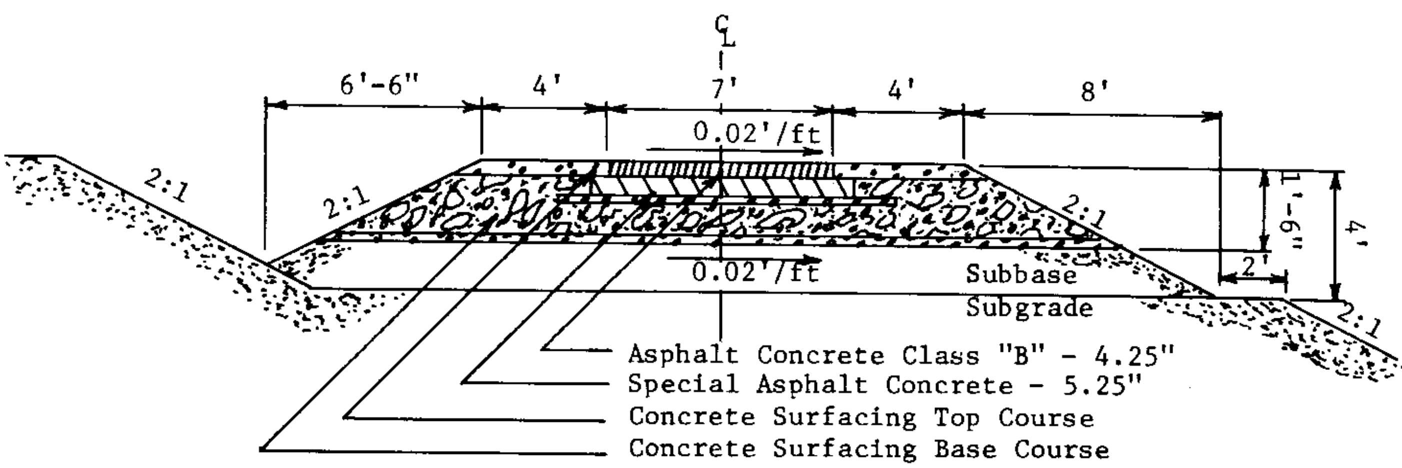
TEST RINGS #2, #3, and #4



PAVEMENT SECTION 1 & 4



PAVEMENT SECTION 2 & 5



PAVEMENT SECTION 3 & 6

FIGURE 3
TYPICAL CROSS-SECTION OF FLEXIBLE PAVEMENT TEST SECTIONS
RING #1

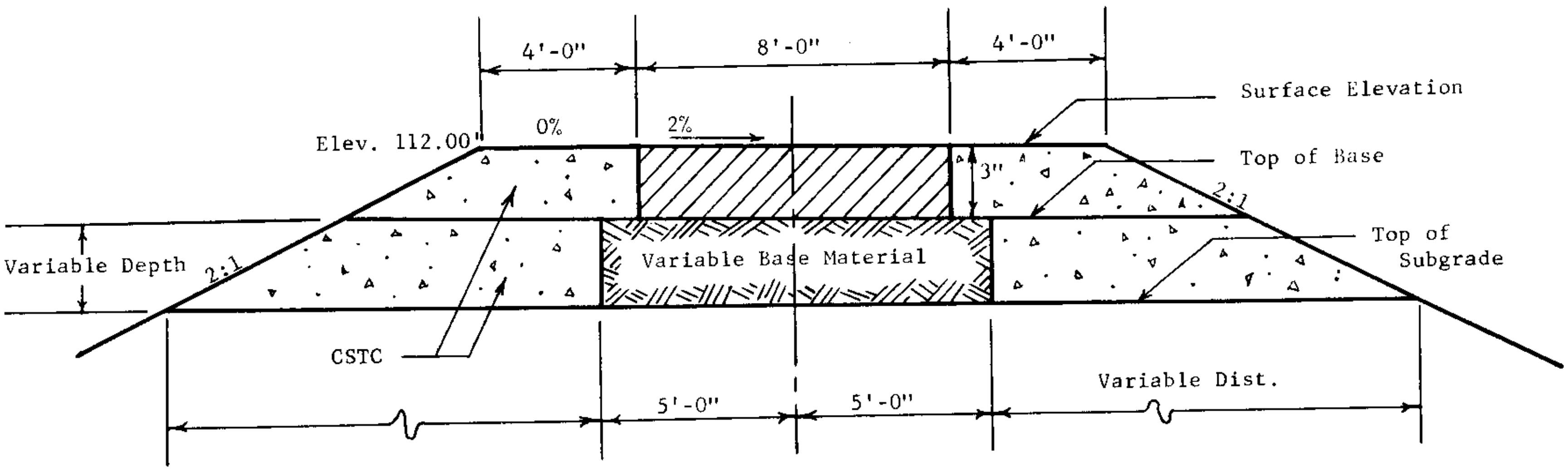


FIGURE 4. TYPICAL CROSS-SECTION OF PAVEMENT STRUCTURE

Rings 2, 3 and 4

TABLE 1: SUMMARY OF TYPES AND THICKNESSES OF EXPERIMENTAL PAVEMENT SECTIONS FOR RING NO. 1

Testing Periods:
March 1, 1965 to May 20, 1966

Wheel Loads: 4,724,100

Course	Material	Section No.	Nominal Thickness Inches
Surface	Class "B" A. C.	All	4.25
Base	Screened Aggregate Cement Treated Base (CTB)	1 & 4	7.75
	Screened Aggregate Asphalt Treated Base (ATB)	2 & 5	5.25
	Class "E" Asphalt Concrete Base (ACB)	3 & 6	5.25
Leveling Course	Crushed Surfacing Top Course - 5/8 inches	1 & 4 2, 3, 5 & 6	1.25 2.50
Crushed Base	Crushed Surfacing Base Course - 1¼ inches	1 & 4 2, 3, 5 & 6	3.25 4.50
Choker Course	Crushed Surfacing Top Course - 5/8 inches	All	1.50
Subgrade	Palouse Loess or Silt	All	10 - 15 feet to bedrock

TABLE 2: SUMMARY OF TYPES AND THICKNESSES OF EXPERIMENTAL PAVEMENT SECTIONS FOR RING NOS. 2, 3 & 4[1]

Ring No.	Testing Periods	Total Wheel Load Applications	Materials, Sections and Nominal Thickness (Inches)				
			Untreated Crushed Surfacing Top Course Base (UTB)	Emulsion Treated Crushed Top Course Base (ETB)	Special Non-Fractured Asphalt Treated Base (ATB)	Sand Asphalt Base (SAB)	Class "F" Asphalt Concrete Base (ACB)
2	10-31-66 to 12-31-66[2] 04-06-67 to 05-31-67[3]	232,608[4]	1 - 4[5] 4.5,7.0,9.5,12.0	5 - 8 3.0,5.0,7.0,9.0	9 - 12 2.0,3.5,5.0,6.5		
3	09-13-67 to 12-04-67[2] 05-10-68 to 07-25-68[3]	870,606[4]	5 - 8 4.5,7.0,9.5,12.0	9 - 12 3.0,5.0,7.0,9.0	1 - 4 0.0,2.0,3.5,5.0		
4	11-06-68 to 12-03-68[2] 04-02-69 to 08-09-69[3]	247,128[4]	9 - 12 4.5,7.0,9.5,12.0			1 - 4 2.0,4.0,6.0,8.0	5 - 8 0.0,2.0,3.5,5.0

[1]Subgrade was Palouse loess or silt for all rings, sections and the depth varied from 10 to 17 feet to bedrock. The surface consisted of 3.0" of Cl "B" asphalt concrete wearing course for all rings.

[2]Fall testing period.

[3]Spring testing period.

[4]This total wheel load application refers to the amount of wheel loads applied on the last surviving test sections.

[5]This refers to section numbers. For example, Section 1 had 4.5" of UTB while Section 4 had 12.0" of UTB.

TABLE 3
PAVEMENT PERFORMANCE SUMMARY - RING #1

Base Type	Section	Base Thickness (inches)	First Appearance of Cracking		Section Failure	
			Date	Wheel Load Applications	Date	Wheel Load Applications
Screened Aggregate Cement-Treated (CTB)	1	7.75	--	--	05-20-66	4,724,100[2]
	4[1]	7.75	06-09-65	2,151,846	06-21-65	2,440,000
Screened Aggregate Asphalt-Treated (ATB)	2[1]	5.25	08-30-65	3,069,720	09-07-65	3,146,000
	5	5.25	--	--	05-20-66	4,724,106[2]
Class "E" Asphalt Concrete (ACB)	3[1]	5.25	08-17-65	2,841,900	09-07-65	3,146,000
	6	5.25	--	--	05-20-66	4,724,100[2]

[1] Artificially saturated after 2 million wheel load applications.

[2] Badly rutted at failure.

TABLE 4

PAVEMENT PERFORMANCE SUMMARY - FALL & SPRING PERIODS

RING #2

Base Type	Section	Base Thickness Inches	First Appearance of Cracking				Section Failure			
			Date	Wheel Load Applications			Date	Wheel Load Applications		
				Fall	Spring	Total		Fall	Spring	Total
Crushed Stone Untreated (UTB)	1	4.5	11-22-66	157,203		157,203	04-13-67	205,425	4,845	210,270
	2	7.0	11-14-66	99,000		99,000	12-05-66	205,425		205,425
	3	9.5	11-14-66	100,000		100,000	03-05-67	205,425	1,323	206,748
	4	12.0	11-18-66	153,411		153,411	05-29-67	205,425	27,183	232,608
Emulsion Treated (ETB)	5	3.0	11-14-66	99,000		99,000	11-30-66	199,233		199,233
	6	5.0	11-14-66	99,000		99,000	11-30-66	199,233		199,233
	7	7.0	11-24-66	175,581		175,581	05-19-67	205,425	25,431	230,856
	8	9.0	12-01-66	197,811		197,811	05-29-67	205,425	27,183	232,608
Special Aggregate Asphalt Treated (ATB)	9	2.0	04-10-67	*	2,874	208,299	04-10-67	205,425	3,471	208,896
	10	3.5	04-25-67	*	5,895	211,320	05-04-67	205,425	16,026	221,451
	11	5.0	04-26-67	*	10,614	216,039	05-29-67	205,425	27,183	232,608
	12	6.5	05-22-67	*	25,779	231,204	11-22-66	*	*	*

*Section did not crack or fail in the time period indicated.

TABLE 5
PAVEMENT PERFORMANCE SUMMARY - FALL & SPRING PERIODS
RING #3

Base Type	Section	Base Thickness Inches	First Appearance of Cracking				Section Failure			
			Date	Wheel Load Applications			Date	Wheel Load Applications		
				Fall	Spring*	Total		Fall	Spring*	Total
Crushed Stone Untreated (UTB)	5	4.5	10-12-67	379,047	--	379,047	11-13-67	679,107	--	679,107
	6	7.0	10-12-67	379,047	--	379,047	11-13-67	679,107	--	679,107
	7	9.5	11-06-67	636,246	--	636,246	12-04-67	735,573	--	735,573
	8	12.0	11-29-67	727,161	--	727,161	07-25-68	735,573	135,033	870,606
Emulsion Treated (ETB)	9	3.0	05-10-68	**	1,164	736,737	05-16-68	735,573	9,150	744,723
	10	5.0	06-22-68	**	37,173	772,746	07-09-68	735,573	58,905	794,538
	11	7.0	--	**	***	--	07-25-68	735,573	135,033	870,606
	12	9.0	--	**	***	--	07-25-68	735,573	135,033	870,606
Special Aggregate Asphalt Treated (ATB)	1	0.0	10-12-67	379,047	--	379,047	11-13-67	679,107	--	679,107
	2	2.0	11-09-67	669,579	--	669,579	06-13-68	735,573	28,824	764,397
	3	3.5	06-21-68	**	34,671	770,244	06-24-68	735,573	48,024	783,597
	4	5.0	--	**	***	--	07-25-68	735,573	135,033	870,606

* To July 25, 1968

** Section did not crack or fail in the time period indicated.

*** These sections did not show any cracking at termination of testing period.

TABLE 6
PAVEMENT PERFORMANCE SUMMARY - FALL & SPRING PERIODS
RING #4

Base Type	Section	Base Thickness (In.)	First Appearance of Cracking				Section Failure			
			Date	Wheel Load Applications			Date	Wheel Load Applications		
				Fall	Spring	Total		Fall	Spring	Total
Sand Asphalt Base (S.A.B.)	1	2.0	4-4-69	1	1,290	144,660	5-20-69	143,370	13,650	157,020
	2	4.0	5-26-69	1	16,419	159,789	5-28-69	143,370	21,420	164,790
	3	6.0	6-18-69	1	32,250[4]	175,620	6-20-69	143,370	34,131[5] 69,621[6]	177,501[5] 247,128[6]
	4	8.0		1			8-9-69	143,370	103,758	247,128
Cl "F" Asphalt Concrete Base (A.C.B.)	5	0.0	11-16-68	47,991		47,391	4-4-69	143,370	1,290	144,660
	6	2.0	4-29-69	1	5,517	148,887	5-22-69	143,370	14,865	158,235
	7	3.5	5-27-69	1	17,892	161,262	6-6-69	143,370	27,798	171,168
	8	5.0		1			8-9-69	143,370	103,758	247,128
Crushed Surfacing Top Course Untreated (U.T.B.)	9	4.5	11-8-68	12,000		12,000	11-10-68	36,681		36,681
	10	7.0	11-16-68	47,391		47,391	11-20-68 4-4-69	104,187[2] 143,370	 1,290	104,187[2] 144,660
	11	9.5	11-16-68	48,000		48,000	4-4-69	143,370	1,290	144,660
	12	12.0	11-16-68	49,104		49,104	4-4-69 8-9-69	143,370	1,290[3] 102,468[6]	144,660[3] 247,128[6]

[1] Section did not crack or fail in the time period indicated.
[2] One-half section 10 was removed.
[3] This section was overlaid with 1-1 1/2" of hot asphalt concrete mix.
[4] Section started to rut very badly.
[5] An asphalt concrete overlay 1 1/2-2 1/2" deep was put on this section.
[6] Wheel loads applied with overlay.

PAVEMENT PERFORMANCE AND EQUIVALENCY SUMMARIES

The results from all four rings have been tabulated into four tables which show the amount of wheel load applications to initial cracking and failure. Tables 3, 4, 5 and 6 show the pavement performance summaries for Rings #1, 2, 3 and 4, respectively. Additional details can be obtained in references (1, 2, 3 and 4).

From the above tables, equivalencies could be calculated for all four rings. Equivalencies from Ring #1 have been calculated and are shown in Table 7. It should be noted that although at the end of testing Ring #1, the surviving sections were badly rutted, excavations revealed that the CTB was fatigue cracked in the wheel path while the asphalt concrete bases were intact. Hence, the equivalencies may perhaps be too high.

Equivalencies based on Rings #2, #3 and #4 are shown in Tables 8 and 9. Table 9 shows the equivalencies in terms of the control material, the crushed surfacing top course untreated base. These results can be used for comparison of materials for design purposes.

TABLE 7
EQUIVALENCIES IN TERMS OF CLASS "E" ACB - RING #1
(4.25 inches of Class "B" A.C. Wearing Course)

Base Type	Saturated Subgrade Conditions		At End of Test
	Initial Cracking	At Failure	Inches
Class "E" ACB*	1.00	1.00	1.00
ATB*	0.93	1.00	1.00
CTB	1.95	1.90	1.50

*Hot-mix

TABLE 8

EQUIVALENCIES BASED ON RINGS #2, #3 & #4

Base Type	Fall[1] Period (in.)	Spring[2] Period (in.)
Crushed Stone Base (UTB)	9.5	12.0
Emulsion Treated Crushed Stone (ETB)	3.0	9.0
Special Aggregate Asphalt Treated (ATB)[3]	2.0	5.0
Class "F" Asphalt Concrete (ACB)[3]	2.0	5.0
Sand-Asphalt Base (SAB)[3]	2.0	8.0

[1]The thinnest sections which survived this period.

[2]The thickest sections which failed during this this period.

[3]Hot mix

TABLE 9

EQUIVALENCIES IN TERMS OF UTB
(3.0" of Class "B" A.C. Wearing Course)

Base Type	Fall Period (in.)	Spring Period (in.)
UTB	1.00	1.00
ETB	0.32	0.75
ATB	0.21	0.42
ACB	0.21	0.42
SAB	0.21	0.67

DESIGN AND OVERLAY PREDICTION FOR ASPHALT PAVEMENTS

The numerous Benkelman Beam rebound deflection data taken over the testing span of Rings #2, #3 and #4 can be utilized to develop a procedure which can be used to predict asphalt pavement life, thickness and overlay need. Most of the Benkelman Beam deflection data used in this procedure has come mainly from Rings #3 and #4, since the data from Ring #2 was incomplete. The procedure used here was mainly developed by Shashi Kant Sharma, when he was on the Highway Research Section staff (8).

Temperatures at various depths in the pavements were taken with all Benkelman Beam rebound deflection readings. These were used to correct the Benkelman Beam data to a temperature base of 70°F. The temperature adjustment factors were developed by The Asphalt Institute (9); the relationship between temperature and adjustment factor is shown in Appendix A. Curve A was derived from granular base pavements and with thick asphalt bases. Curve B was used in the present study due to the thinness of the asphalt section which has a larger temperature variation. An average curve was developed for the emulsion treated base; this is curve C in Appendix A.

The wheel load applications applied on the test track were from a 10,500 pound load on three sets of dual tires, each of which was equivalent to a 21,000 pound single axle load. To conform with the American Association of State Highway Officials (AASHO) design standards (10), these wheel load applications were equated to 18,000 pound single axle load equivalencies. This evaluation was done by establishing wheel load equivalencies determined by AASHO test road results in Appendix A, which shows the relationship between wheel load equivalency factor and axle wheel load.

The table in Appendix A shows a sample calculation for temperature correction and equivalent wheel load applications for section 1, Ring #3, which had no base. These calculations were done for all sections with various bases and thicknesses in all rings.

Individual graphs from these tables were plotted for equivalent wheel load applications and Benkelman Beam deflections at a constant base thickness. The least squares method was used to draw the curves. Some typical curves can be found in Appendix B. These curves generally follow a certain pattern. Initially, they are nearly horizontal, followed by a concave rise and a convex portion before they fail. In some cases, the thin pavements carried more EWL applications; this may be attributed to different conditions of construction and environment. Some of the thin pavements were built under slightly different construction conditions and had some differences in environment which resulted in unequal EWL applications.

Another set of curves were developed from the above graphs in which the values of Benkelman Beam deflections and thicknesses of the pavement were plotted; here the equivalent wheel load (EWL) applications were kept constant. These sets of curves are shown in Figures 5-8. These curves have no factor of safety and one should be selected depending upon the judgment of the design engineer.

Benkelman Beam rebound deflections were taken during the appearance of cracks and at failure. All these readings were corrected to a standard temperature of 70^{o}F. Since the testing periods were in the fall and spring, the subgrade moisture contents and their effects were assumed to be the same at any time period.

Certain terms have to be defined as they will be used in the forthcoming discussion.

Construction Design Deflection (CDD): This is defined as that deflection which is measured after the completion of construction, and which may be taken as a factor for the performance control of the contractor. Those deflections are shown in Figures 5-8, and are tabulated in Table 10.

Service Design Deflection (SDD): This is defined as that deflection at which an overlay is needed.

Failure Design Deflection (FDD): This is defined as that deflection at which the road will be declared to have failed.

In many sections the cracks occurred somewhat earlier than their contemporary sections; some showed very little loading between the occurrence of cracks and failure. The data for corrected deflection at which cracks occurred and for failure deflections are tabulated in Tables 11 and 12. A percentage ratio was taken between the cracking deflection and failure deflection, and was designated as "pavement performance".

$$\text{Pavement Performance} = \frac{\text{Deflection at Cracking}}{\text{Deflection at Failure}} \times 100$$

From the pavement performance, a conservative approach was made on the assumption that between the first appearance of cracks and the final failure, an overlay will be needed. A pavement performance based on this discussion is tabulated in Table 11 and mean pavement performance for different types of pavements is tabulated in Table 13.

Using three-dimensional geometry and the theory of interpolation, two equations can be obtained which can be used for thickness design and estimated wheel load life. The mathematical proof for these equations is given in Appendix C.

These equations are:

$$T_e = T_f \left[1 + \frac{W_e - W_f}{W_f} \right] \qquad \text{When } D_e = D_f \qquad (A)$$

and

$$W_f = W_e \left[1 + \frac{D_f - D_e}{D_e} \right] \qquad \text{When } T_e = T_f \qquad (B)$$

where: W_f = EWL application at failure

T_f = Thickness of the base

D_f = Benkelman Beam rebound deflection at failure

W_e = Expected EWL applications

T_e = Expected thickness of pavement

D_e = Expected deflection at W_e and T_e

These two equations are to be used in conjunction with Figures 5 - 8. Equation (A) will give the expected safe thickness for expected EWL applications depending upon the data read off from Figures 5 - 8. Equation (B) will give the number of EWL applications at which the pavement may fail depending upon the Benkelman Beam deflection, when the thickness of the pavement is known and constant.

TABLE 10

CONSTRUCTION DESIGN DEFLECTIONS FOR PAVEMENTS

Section Number	Type	Thickness (in.)	Measured Construction Design Deflection (inches)
1	Special Aggregate Asphalt Treated Base (ATB)	0.0	.037
2		2.0	.034
3		3.5	.031
4		5.0	.013
5	Untreated Crushed Rock Base (UTB)	4.5	.038
6		7.0	.037
7		9.5	.034
8		12.0	.031
9	Emulsion Asphalt Treated Base (ETB)	3.0	.037
10		5.0	.034
11		7.0	.027
12		9.0	.016

TABLE 11

MEASURED DEFLECTION AT CRACKING AND FAILURE FOR TEST RING #3

Section	Type	Thickness (inches)	Occurrence of Cracks at Pavement Temperature	Failure at Pavement Temperature	Occurrence of Cracks at 70°F	Failure at 70°F	Deflection at Cracking / Deflection at Failing (%)
1	ATB[1]	0.0	.060	.061	.05880	.1024	57.4
2	ATB	2.0	.054	.125	.04664	.1230	37.8
3	ATB	3.5	.047	.140	.03136	.1180	26.5
4	ATB	5.0	.027	.081	.03780	.0496	63.4
5	UTB[2]	4.5	.043	.076	.04230	.0592	71.4
6	UTB	7.0	.068	.111	.06490	.0920	70.5
7	UTB	9.5	.057	.084	.05890	.0907	64.9
8	UTB	12.0	.706	.085	.07020	.0808	86.8
9	ETB[3]	3.0	.038	.093	.05510	.1212	45.5
10	ETB	5.0	.031	.129	.04270	.1136	37.5
11	ETB	7.0	.049	.078	.06200	.0772	80.3
12	ETB	9.0	.052	.0857	.06430	.0762	84.3

[1] ATB: Special aggregate asphalt treated base

[2] UTB: Untreated crushed rock base

[3] ETB: Emulsion asphalt treated base

TABLE 12

MEASURED DEFLECTION AT CRACKING AND FAILURE FOR TEST RING #4

Section	Type	Thickness (inches)	Occurrence of Cracks at Pavement Temperature	Failure at Pavement Temperature	Occurrence of Cracks at 70°F	Failure at 70°F	Deflection at Cracking / Deflection at Failing (%)
1	SAB[1]	2.0	--	--	--	--	--
2	SAB	4.0	--	--	--	--	--
3	SAB	6.0	.1711	.1116	.1350	.1530	88.2
4	SAB	8.0	.0630	.1240	.0660	.1059	62.6
5	ATB[2]	0.0	.0424	.2070	.0848	.2173	38.9[4]
6	ATB	2.0	.1280	.1590	.1200	.1220	98.3
7	ATB	3.5	.0430	.1440	.0756	.1230	61.4
8	ATB	5.0	--	--	--	--	--
9	UTB[3]	6.5	--	--	--	--	--
10	UTB	7.0	.0680	.1540	.0830	.1530	54.2
11	UTB	9.5	.0740	.1360	.0910	.1350	67.4
12	UTB	12.0	.0796	.0979	--	--	--

[1]SAB: Sand Asphalt Base
[2]ATB: Asphalt Treated Base, Class "F"
[3]UTB: Untreated Crushed Rock Base

[4]38.9: This section bridged the fall and spring test periods; the cracks started in the fall and the section failed in the spring, hence, the deflections were lower in the fall.

TABLE 13

PAVEMENT PERFORMANCE FACTORS FOR OVERLAY PREDICTIONS AT SERVICE DESIGN DEFLECTION

Type	Pavement Performance Factor
Special Aggregate Asphalt Treated Base	60%
Untreated Crushed Rock Base	70%
Emulsion Asphalt Treated Base	80%
Sand Asphalt Base	80%
Asphalt Treated Base, Class "F"	90%

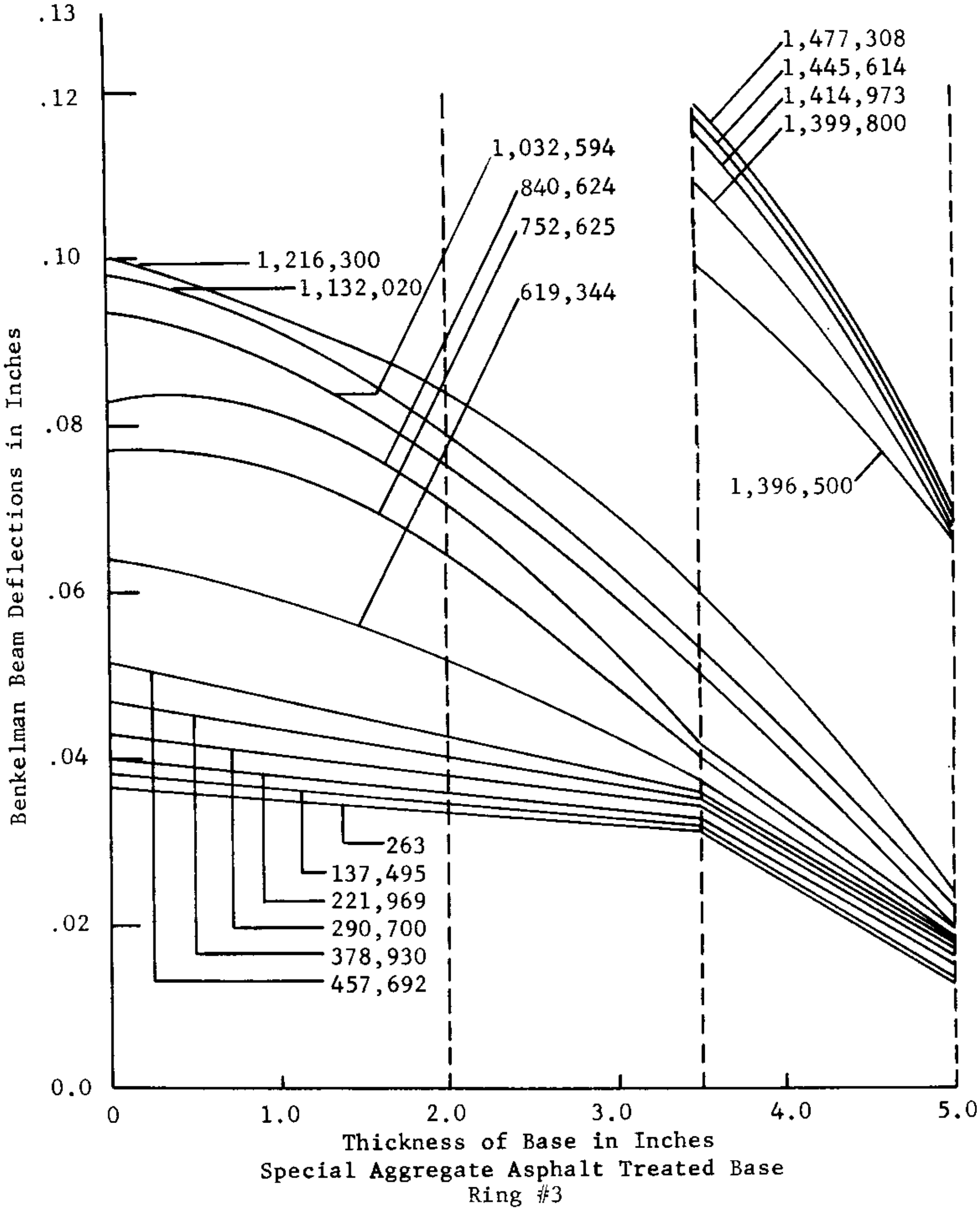


FIGURE 5
RELATION BETWEEN DEFLECTION AND BASE THICKNESS FOR VARIOUS EWL APPLICATIONS.

Note: All the base thicknesses are covered with 3.0 inches of wearing course.

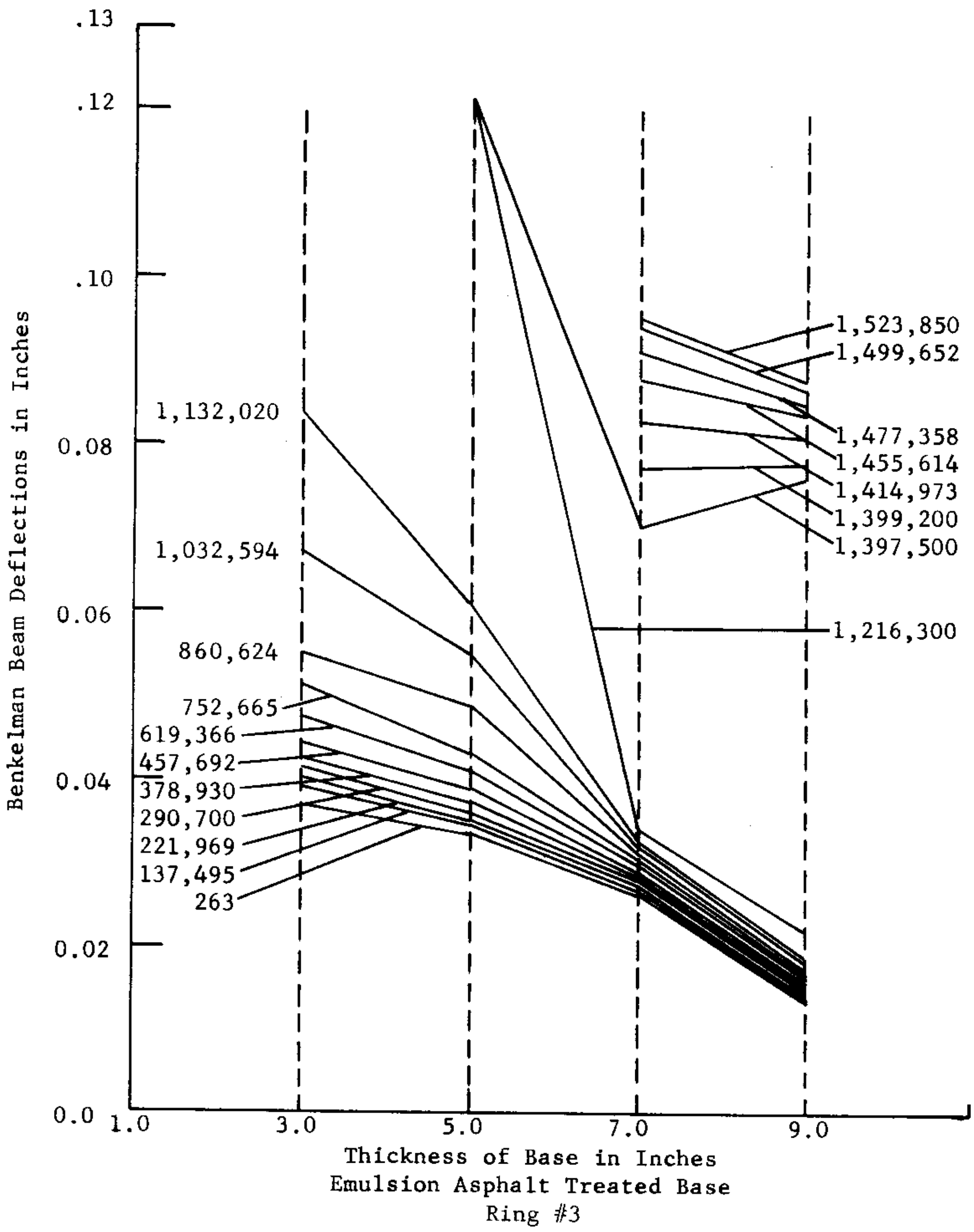
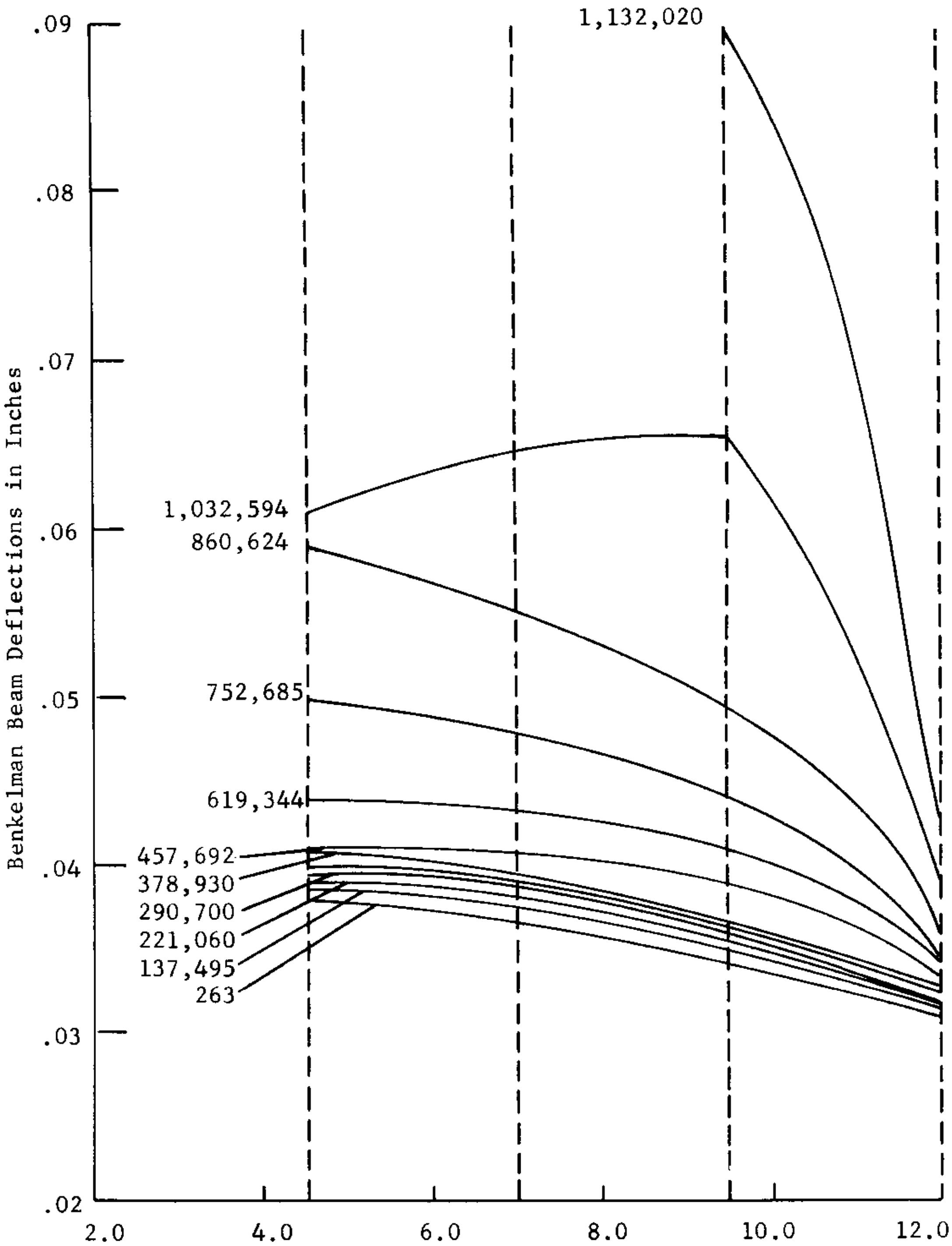


FIGURE 6
RELATION BETWEEN DEFLECTION AND BASE THICKNESS FOR VARIOUS EWL APPLICATIONS.



Thickness of Base in Inches
Untreated Crushed Rock Base
Ring #3

FIGURE 7

RELATION BETWEEN DEFLECTION AND THICKNESS FOR VARIOUS EWL APPLICATIONS.

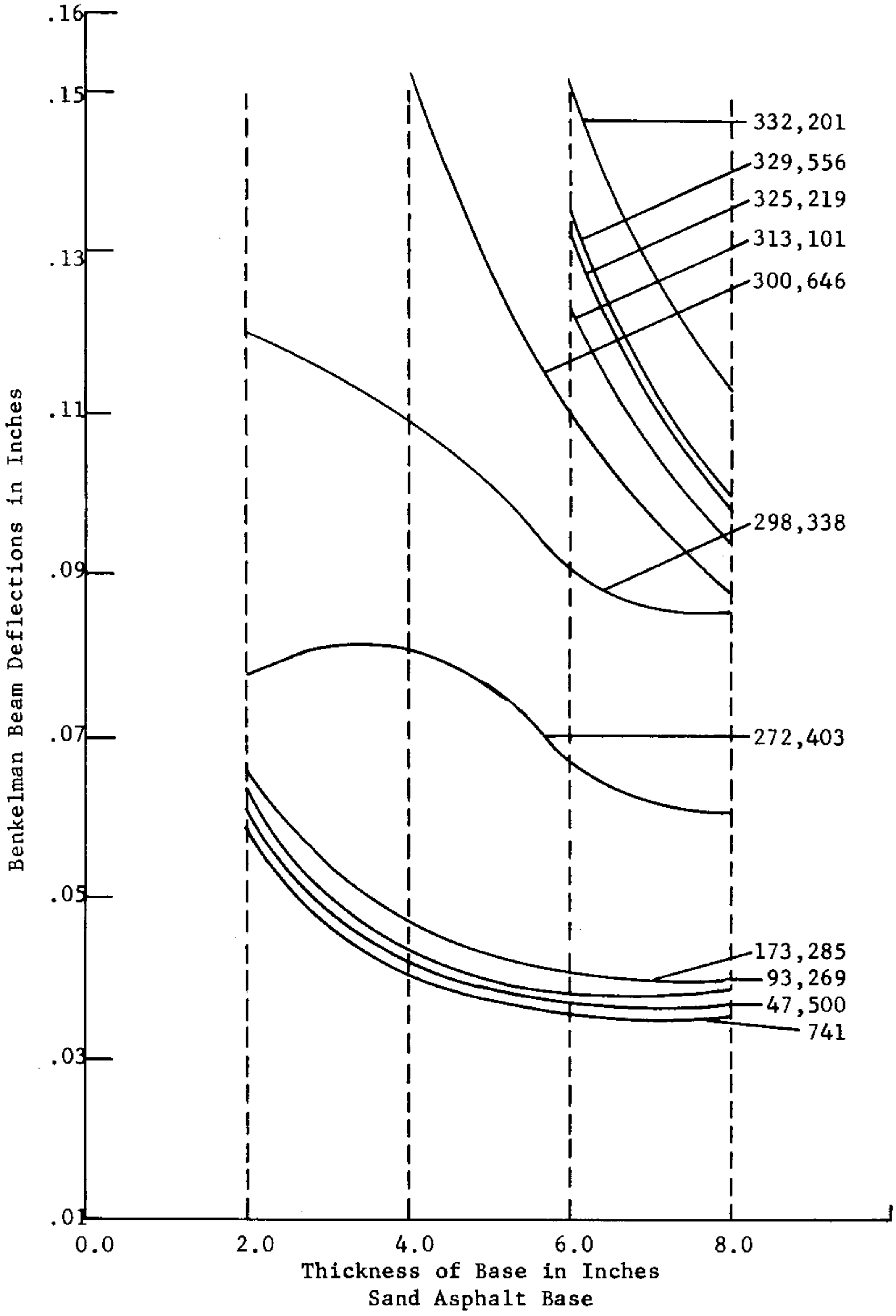


FIGURE 8
RELATION BETWEEN DEFLECTION AND THICKNESS FOR VARIOUS EWL APPLICATIONS.

EXAMPLE AND SUMMARY

Overlay Prediction After Cracking

For all the pavements which are built on Palouse soil or on a soil which is similar to it in characteristics and are constructed with the constituents conforming to the specifications given in the test track construction reports, the following method may be adopted.

Assume the following: A pavement has a five-inch thickness of emulsion asphalt treated base, the mean temperature of the pavement is 50°F and the axle wheel load is 18,000 pounds.

Then for such a pavement the Failure Design Deflection is 0.12 inches as in Figure 6. The pavement performance factor for such a pavement is 80% as in Table 13.

Thus Service Design Deflection (SDD):

SDD = Pavement Performance Factor x Failure Design Deflection

= 0.80 x 0.12 inches

= 0.096 inches at 70°F and for 18,000 pounds axle wheel load

When the temperature is 50°F the SDD for 18,000 pounds EWL will be:

SDD = 0.096 x 1/1.38

SDD = 0.0696 inches or 0.07 inch

Thus when the Benkelman Beam deflection reaches 0.07 inches, an overlay may be placed on the road. The thickness of the overlay is at the discretion of the engineer in charge. It should be noted that any change in the axle wheel load will not change the SDD, because the SDD is a property of the pavement material. A low axle wheel load represents a smaller EWL, and contributes less deterioration to the pavement than a heavy wheel load, and therefore, the SDD is approached with a lower number of heavy wheel loads than with light ones.

The temperature correction adjustment factor is taken from Appendix A.

Overlay Prediction Before Cracking

An overlay could also be placed just before the cracks appear. This could be done by use of the values obtained at the test track and are shown in Table 11.

EXAMPLE: There is a pavement system with a seven inch thick layer of untreated crushed rock base on the Palouse soil (Table 11), it will have a deflection equal to 0.0649 inches at 70°F.

An overlay could be placed at 0.06 inches of Benkelman Beam deflection. The judgment for such a decision depends entirely upon the engineer in charge as no hard and fast rules can possibly be made. It is suggested that if an overlay was considered to be required at the point of 90% of the deflection shown in the table, the overlay could be placed just before the appearance of the cracks. The 90% value will give a factor of safety of 1.11.

Design of Pavement when EWL is Known

Suppose 1,000 equivalent wheel loads per day of 18,000 pounds axle load are expected and the road is expected to be in use for three years, then the total EWL expected are:

$$EWL = 1{,}000 \times 365 \times 3$$

$$EWL = 1{,}095{,}000$$

An approximate wide choice can be made from Figures 5 - 8, such as:

2-inch thick special aggregate asphalt treated base, 5-inch thick emulsion asphalt treated base or 9.5-inch thick untreated crushed rock base.

When EWL and Failure Design Deflection are known, another approach is to assume EWL is 1,095,000 and FDD is 0.12 inch.

From Figure 6, one can see that a five-inch thick emulsion asphalt treated base can withstand 1,216,300 EWL at this FDD. The reduction of thickness can result in having the same deflection at reduced EWL. Thus by the formula discussed above and in Appendix C, we have.

$$T_e = T_f \left[1 + \frac{W_e - W_f}{W_f}\right]$$

$$T_e = 5.0 \left[1 + \frac{1,095,000 - 1,216,300}{1,216,300}\right]$$

$$T_e = 5.0 \left[1 + \frac{121,300}{1,216,300}\right] = \left[1 - .0997\right]$$

$$T_e = 5.0 \times 0.9003$$

$$T_e = 4.5 \text{ inches of EATB}$$

When Failure Design Deflection and thickness are known; supposing the FDD is 0.12 inch and the pavement is 5.0 inches thick of special aggregate asphalt treated base, the allowable EWL can be determined before it reaches Failure Design Deflection. Using the second formula shown in Appendix C, one proceeds as follows:

$$W_f = W_e \left[1 + \frac{D_f - D_e}{D_e}\right]$$

From Figure 5, one measures that at 0.07 inch of Benkelman Beam deflection the EWL is 1,477,308. Then the EWL at Failure Design Deflection will be:

$$W_f = 1,477,308 \left[1 + \frac{0.12" - 0.07"}{0.07}\right]$$

$$W_f = 1,477,308 \ [1 + 0.7142] = 2,532,401$$

Thus, 2,532,401 EWL can be applied on a five-inch thick special aggregate asphalt treated base pavement before it reaches its Failure Design Deflection.

It must be noted that the above results are at 70°F and for 18,000 pounds axle load. Any change must be adjusted for temperature and EWL according to Appendix A.

The graphs developed may be of some limited use in areas having similar soil conditions. If used in other areas with better soils, the results obtained will be conservative and this can be corrected by using a lower factor of safety. It should be remembered that deflections are not necessarily a function of wheel loads and that environment may be more important. Hence, environmental factors may have to be used for different areas.

ANALYSIS OF WASHINGTON STATE UNIVERSITY TEST TRACK

Scope of Analysis

Laboratory data for materials used in the various pavements was used with n-layer elastic theory to predict field response to loading. Of primary interest were the highly instrumented sections in the three rings. The successful prediction of field behavior led to the computation of selected stresses, strains and deflections for all 12 sections. Assuming suitable criteria for the pavement component behavior, equivalent thicknesses for each type of base material was computed and compared to the equivalencies determined from field observation. Analysis of Ring #2 has been done by Terrel (11) and reported (12).

Materials and Laboratory Data

Subgrade

The subgrade was a uniform silt, HRB Classification A-6(10) throughout the four test rings. Moisture density curves and general classification data have been reported (1, 2, 3, 4), but Kallas (13) has conducted tests of special interest to the analysis made herein. This data indicates that field and laboratory conditions were matched reasonably well thus allowing the use of laboratory results directly.

The subgrade material was also tested using conventional California Bearing Ratio (CBR) Techniques and these results are shown in Table 14.

Resilient modulus tests were conducted on the subgrade material over a range of moisture conditions (13). These repeated load triaxial compression

TABLE 14

CALIFORNIA BEARING RATIO (CBR) TEST ON PALOUSE SILT SUBGRADE SOIL

Water Content (%)	Dry Density (lb/cu ft)	CBR (%)	Swell (%)	Water Content After Soaking (%)
		SERIES 1		
13.0	105.1	4.6	2.4	20.2
16.4	108.0	9.2	0.8	18.9
19.3	105.8	2.8	0.3	19.9
		SERIES 2		
13.0	114.0	13.5	1.5	16.4
16.4	112.5	7.6	0.5	17.3
19.3	106.6	2.2	0.5	19.6

Note: Specimens soaked four days, 10 lb surcharge weight.

Series 1 Compaction: 10 lb hammer, 18-in. drop, 5 layers, 12 blows per layer (12,200 ft-lb/ft^3)

Series 2 Compaction: 10 lb hammer, 18-in. drop, 5 layers, 29 blows per layer (26,400 ft-lb/ft^3).

(After Kallas)

tests on a 6-inch diameter by 12-inch high specimens were fabricated using a drop hammer and compacted in 12 layers. Deviator stresses (σ_d) or the repeated load, varied from 1.5 to 12 psi and confining pressures (σ_3) from 0 to 4 psi. The special nature of this subgrade made normal convenient method of plotting results less useful. However, Figures 9 and 10 show the relationship for resilient modulus (M_R) vs. deviator stress for a range of soil conditions and at stress ratios (σ_d/σ_3) of 1.5 and 3.0, respectively. These curves have been used to assist in the selection of approximate subgrade moduli for the analysis of Rings #2, #3 and #4. A more useful relationship between M_R and load stresses is shown in Figure 11; here the bulk stress (θ) or first stress invariant (sum of principal stresses: $\theta = \sigma_1 + \sigma_2 + \sigma_3$) was plotted vs. M_R. This graph was used as a guide to selecting suitable values of M_R in computations.

Since the subgrade is frequently at above optimum moisture, tests by the Asphalt Institute (14) were run on Shelby tube samples obtained from Ring #4. For each sample, M_R was determined for 6 psi deviator stress level, 4 psi and 2 psi confining pressures after 10,000 load repetitions; then M_R was measured for 12 psi deviator stress level, 8 psi and 4 psi confining pressures after 2,000 additional load repetitions; and finally M_R was measured for 6 psi and 12 psi deviator stress levels, 0 psi confining pressure after 300 load repetitions. The results have been plotted in Figure 12 as bulk stress vs. M_R. This graph was also used as a guide to selecting suitable values of M_R in computations for all rings.

<u>Untreated Base</u>

Base courses of untreated crushed basaltic rock (UTB) were used as control sections in Rings #2, #3, and #4. Table 2 shows where UTB was used in

FIGURE 9
RESILIENT MODULUS VS. DEVIATOR STRESS RELATIONSHIP
FOR PALOUSE SILT FOR STRESS RATIO $\frac{\sigma_d}{\sigma_3} = 1.5$

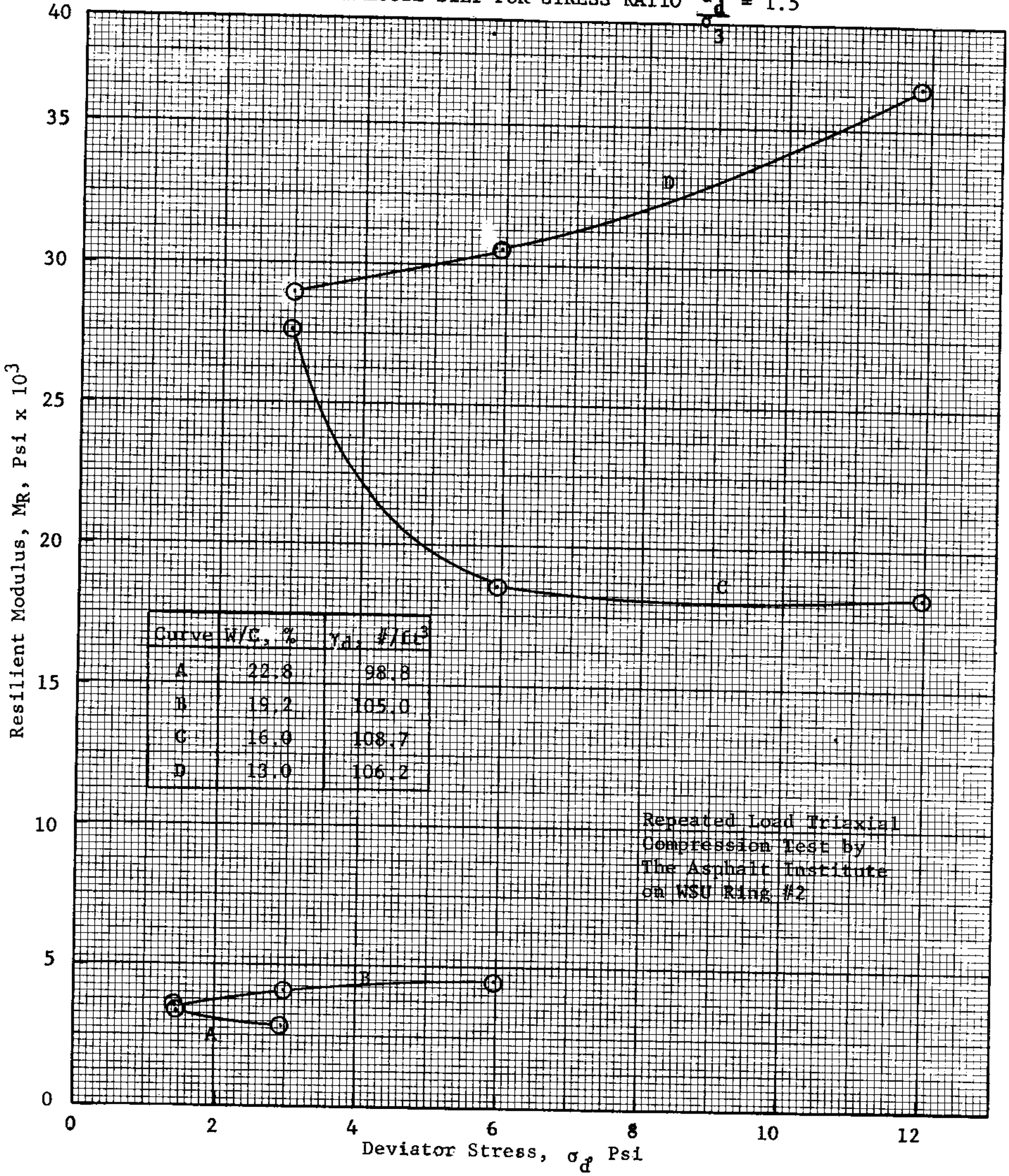


FIGURE 10
RESILIENT MODULUS VS. DEVIATOR STRESS RELATIONSHIP
FOR PALOUSE SILT FOR STRESS RATIO $\frac{\sigma_d}{\sigma_3} = 3.0$

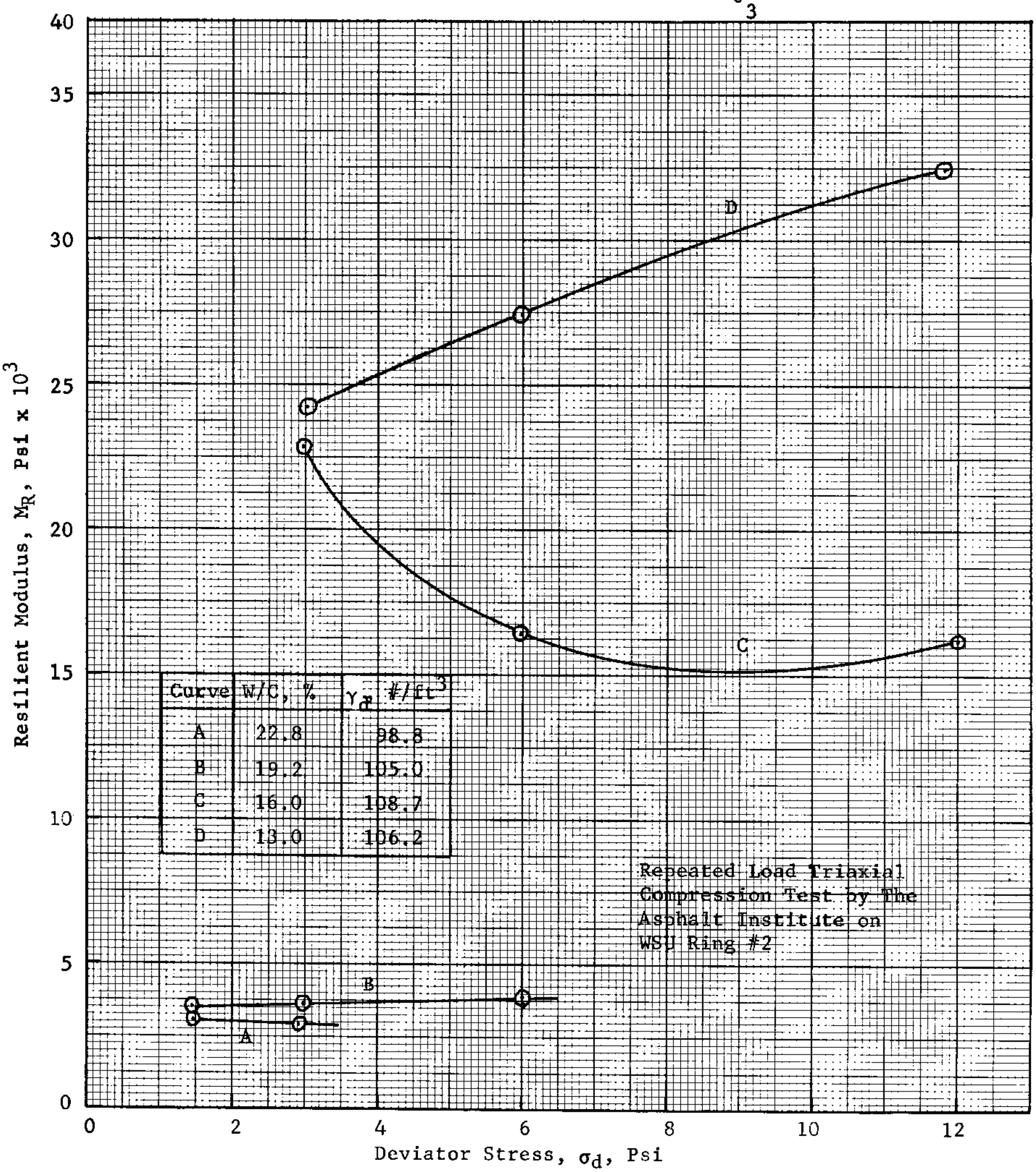


FIGURE 11
RESILIENT MODULUS VS. BULK STRESS RELATIONSHIP FOR PALOUSE SILT

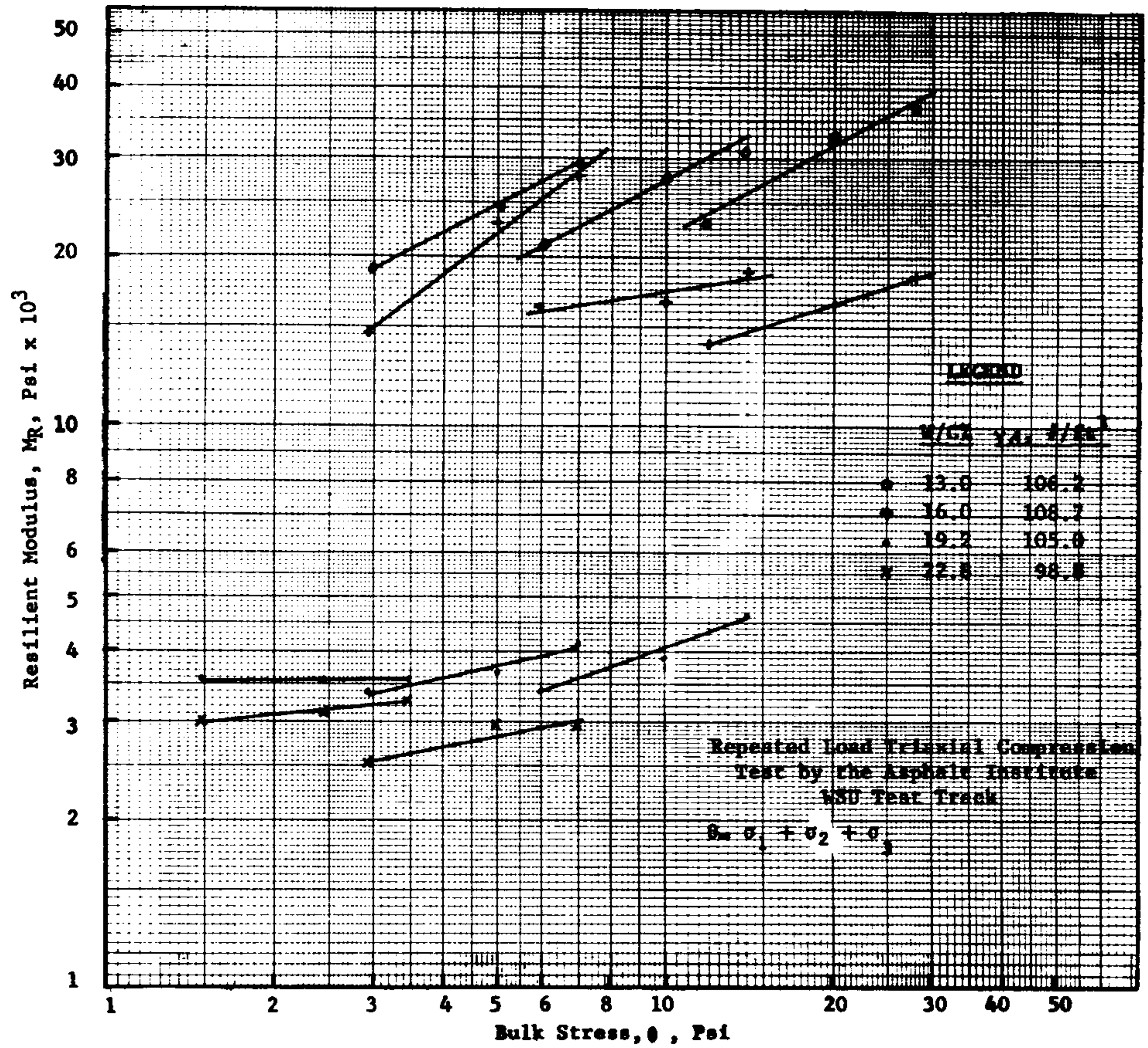
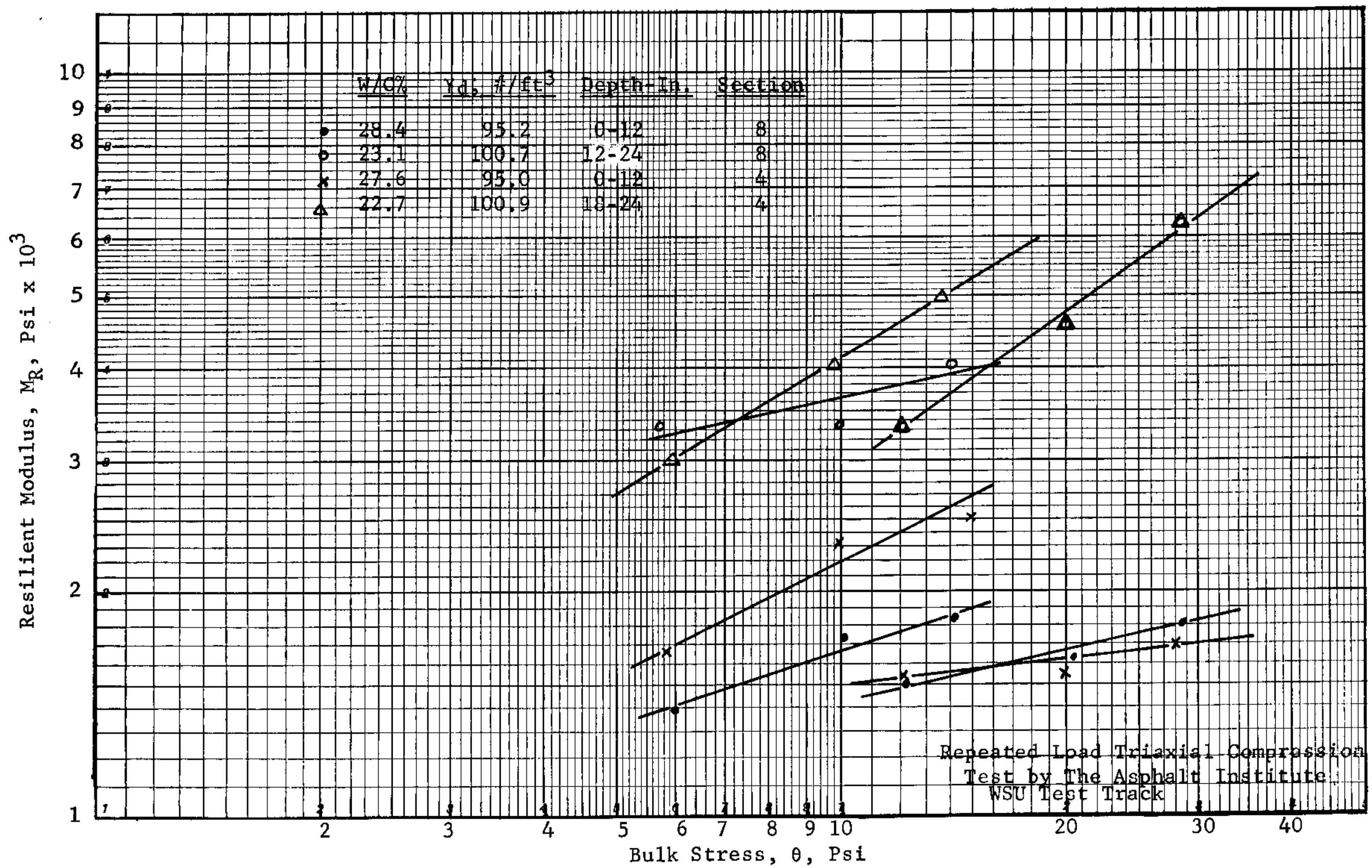


FIGURE 12
RESILIENT MODULUS VS. BULK STRESS RELATIONSHIP FOR PALOUSE SILT - RING #4



in the different rings. Details about the material, such as specifications, gradation and density are given in references (2, 3 and 4).

Repeated load triaxial compression tests were conducted on the untreated base material (13). Specimens measured 6 inches in diameter by 12 inches high and were compacted in 12 layers using a drop hammer. Densities and gradation of this material are in references (2, 3 and 4). Deviator stresses (σ_d) ranged from approximately 15 to 60 psi and confining pressure from 0 to 40 psi. The deviator stress was applied every 0.5 sec. for 0.1 sec. duration and the modulus values reported were measured after 10,000 repetitions.

Figures 13 and 14 show the results of the above tests; the former shows the relationship between resilient modulus (M_R) and confining pressure (σ_3) for material tested under three conditions, while the latter figure shows the same data, but M_R is plotted vs. bulk stress (θ_1). The curves in Figure 14 are more linear and hence more useful for analysis.

Emulsion Treated Base

Asphalt emulsion treated base (ETB) was used in Rings #2 and #3 as shown in Table 2. Aggregate for this base type was identical to that used in the untreated base as described above and elsewhere (2,3,4). The densities and gradation are shown in references (2 and 3). Problems with the mix design of ETB developed in Ring #2 (2) which were corrected in Ring #3(3).

Laboratory testing accomplished by The Asphalt Institute on the material was similar to that for the untreated base as described above. Five percent of SS-Kh asphalt emulsion was used for laboratory-prepared specimens. Repeated load triaxial tests were conducted at two conditions: (1) uncured; immediately

FIGURE 13
RESILIENT MODULUS VS. CONFINING PRESSURE RELATIONSHIP
FOR UNTREATED BASE MATERIAL

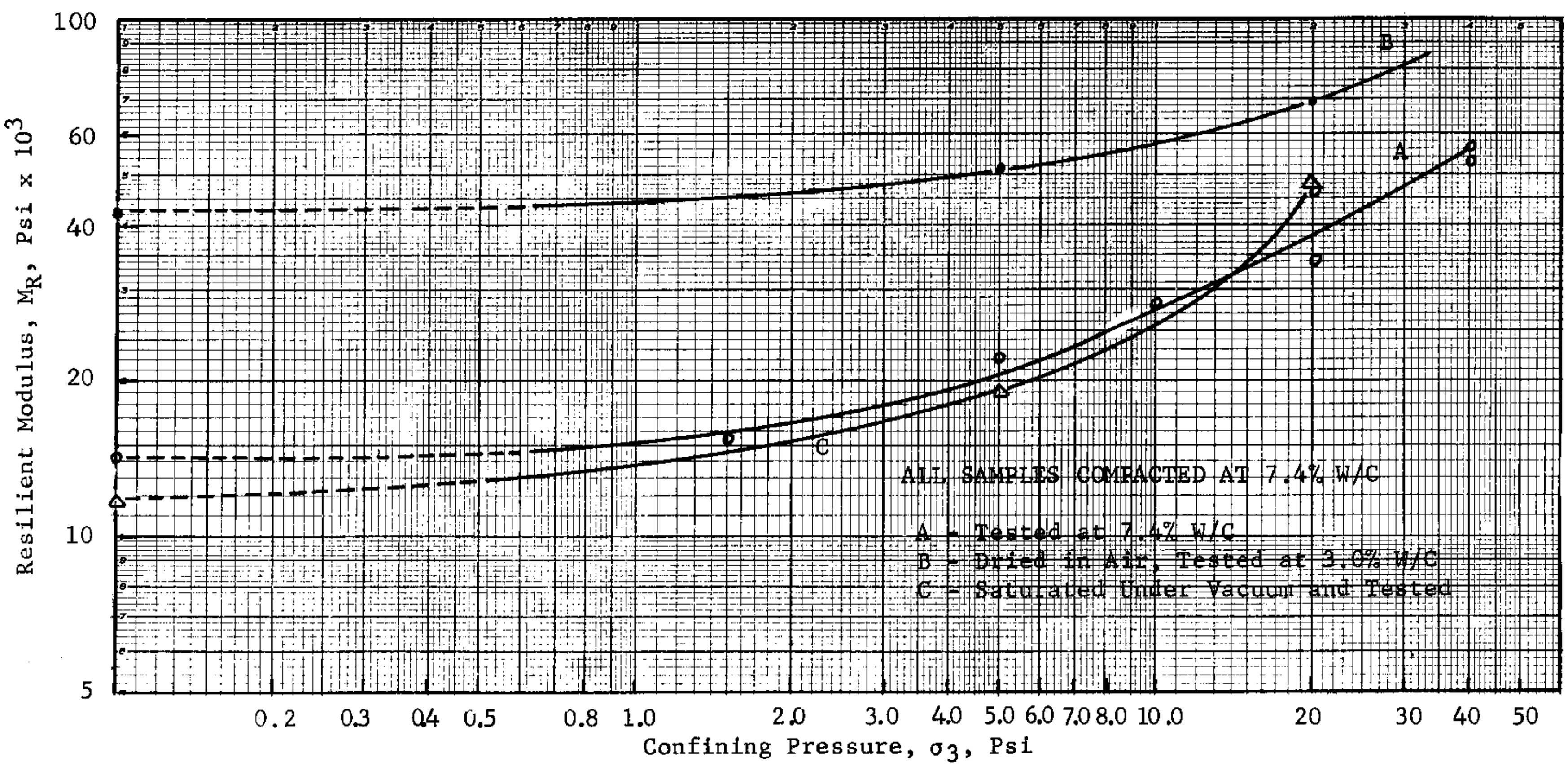
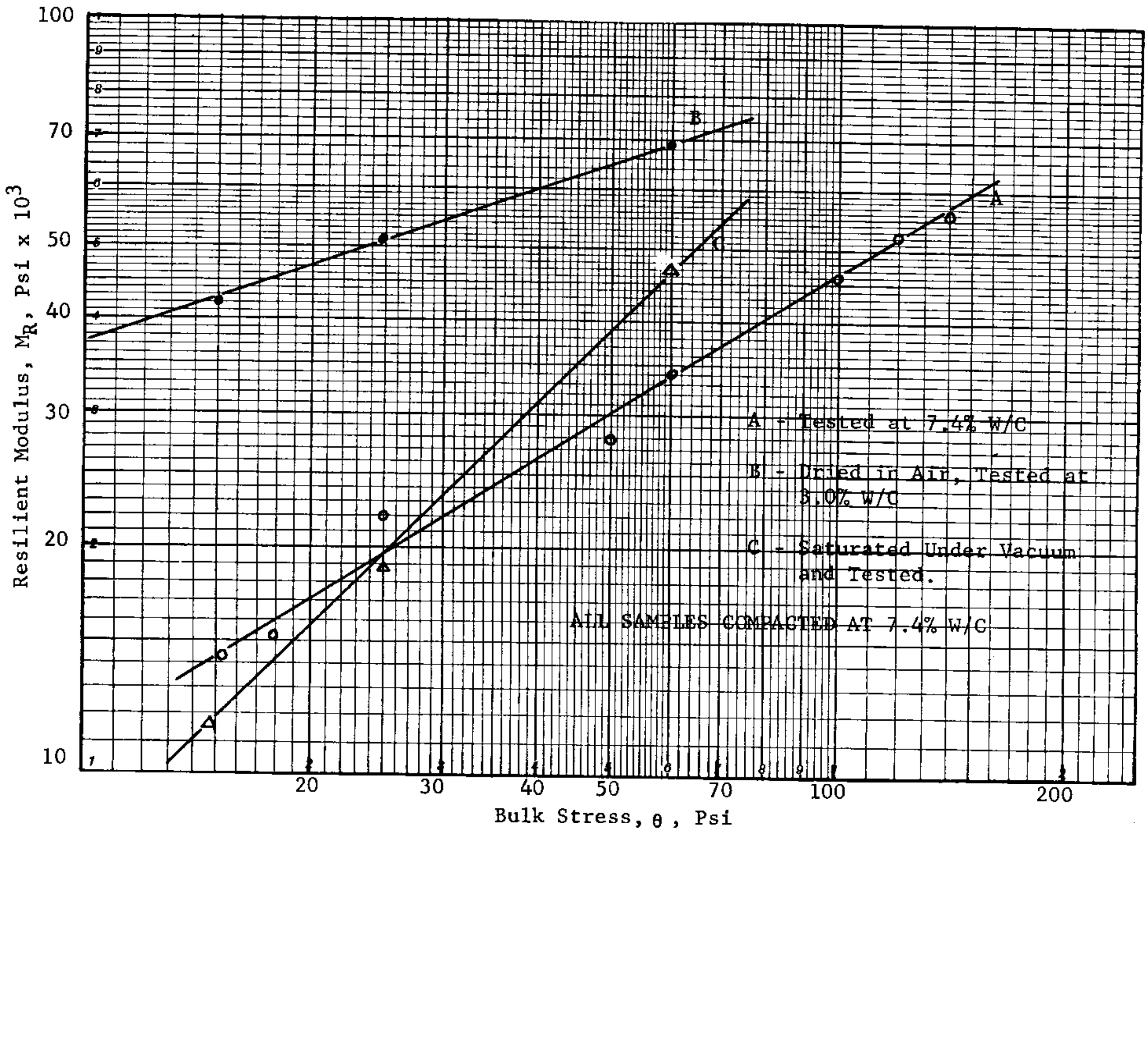


FIGURE 14
RESILIENT MODULUS VS. BULK STRESS RELATIONSHIP
FOR UNTREATED BASE MATERIAL



following compaction, and (2) after six months storage in sealed plastic bags. Figures 15 and 16 show the results of these tests.

Asphalt Treated Base

The aggregate for the Special Asphalt Treated Base was uncrushed gravel which was blended from two sources in eastern Washington. The material was in Rings #2 and #3 and in sections as shown in Table 2. The mix design specifications and densities were discussed in detail in references (2 and 3). Construction was accomplished in a manner similar to that for normal hot mix asphalt paving.

Laboratory-prepared specimens of this material were used in dynamic complex modulus tests (13). Specimens were 4 inches in diameter by 8 inches high and were compacted to average densities. In this type of test, the complex modulus, /E*/ is determined by dividing the sinusoidal stress function (axially applied) by the resulting sinusoidal strain. Three frequencies of stress application (1, 4 and 16 cps) and three temperatures (40, 70 and 100°F) were used in combination with a single stress level (35 psi) to describe this material's response under dynamic loading. Figure 17 shows the results of these tests.

Asphalt Concrete Base

The aggregate for the Class "F" asphalt concrete base is a crushed basalt with a wider gradation range than for the other classes of asphalt concrete specified by the Washington State Highway Department Specifications. This was used in Ring #4. The gradation met the specification for Class "F" aggregate; the gradation density and mix design is in reference (4). The

FIGURE 15
RESILIENT MODULUS VS. CONFINING PRESSURE RELATIONSHIP FOR SS-Kh
EMULSION TREATED BASE

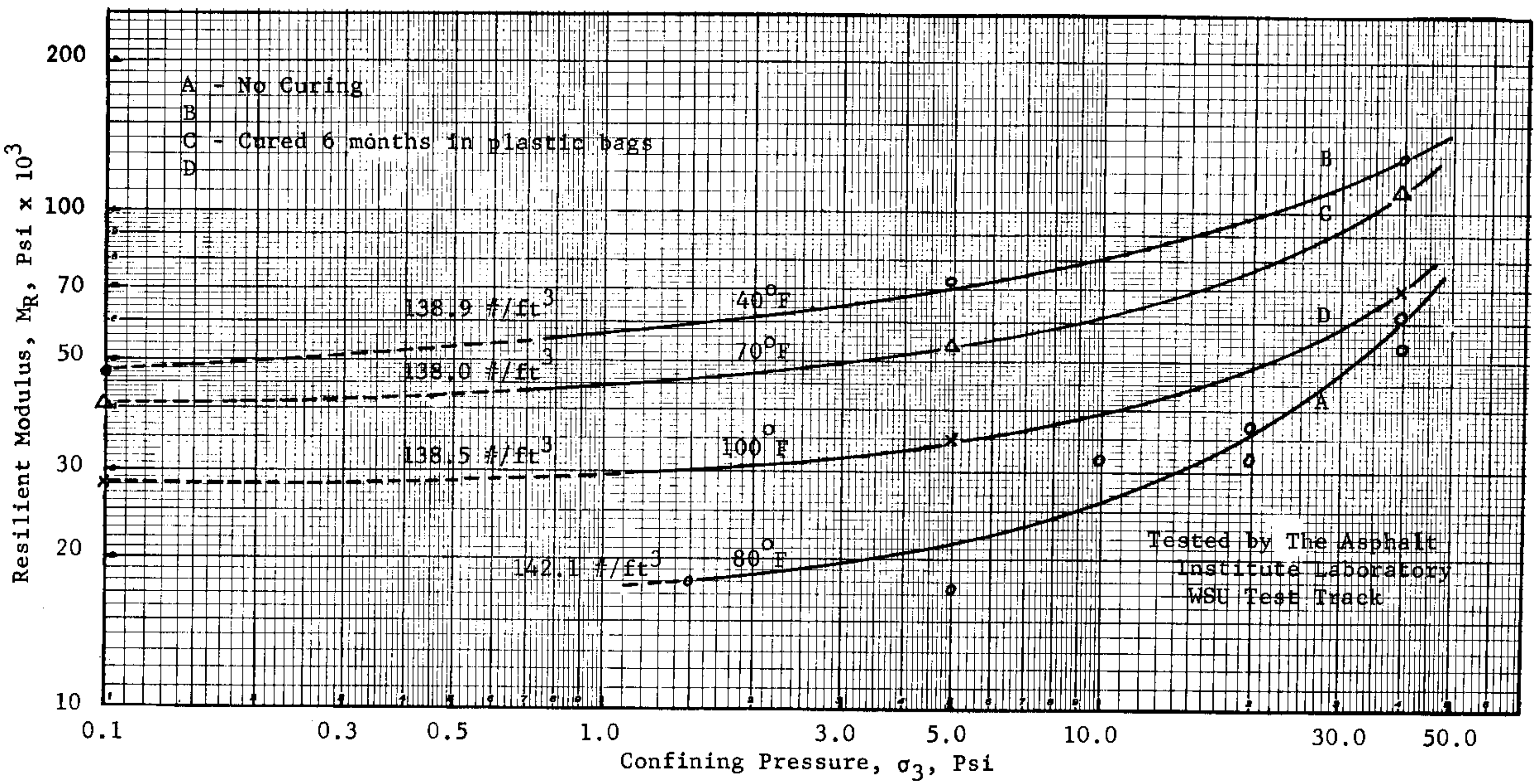


FIGURE 16
RESILIENT MODULUS VS. BULK STRESS RELATIONSHIP
FOR SS-Kh EMULSION TREATED BASE

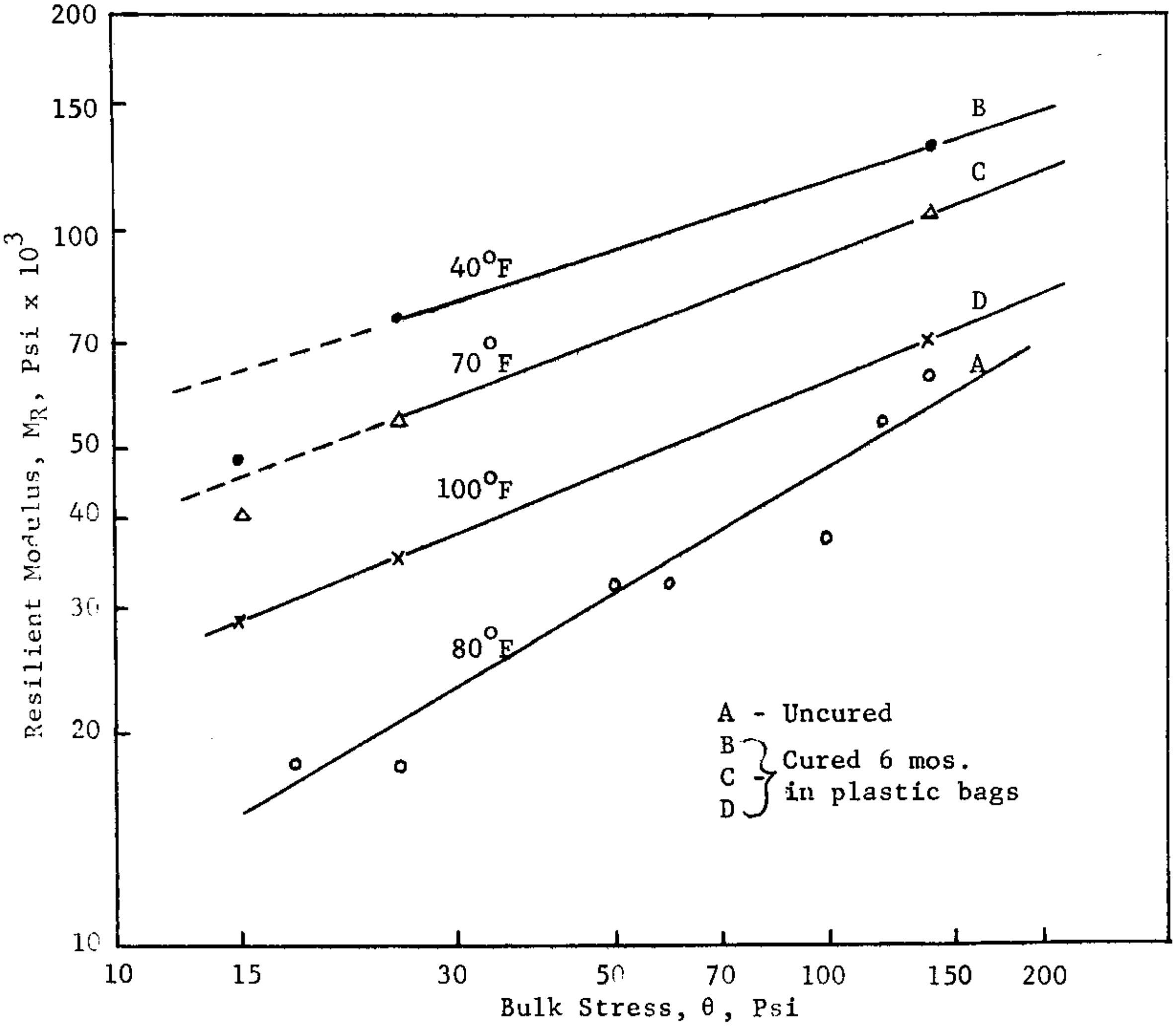
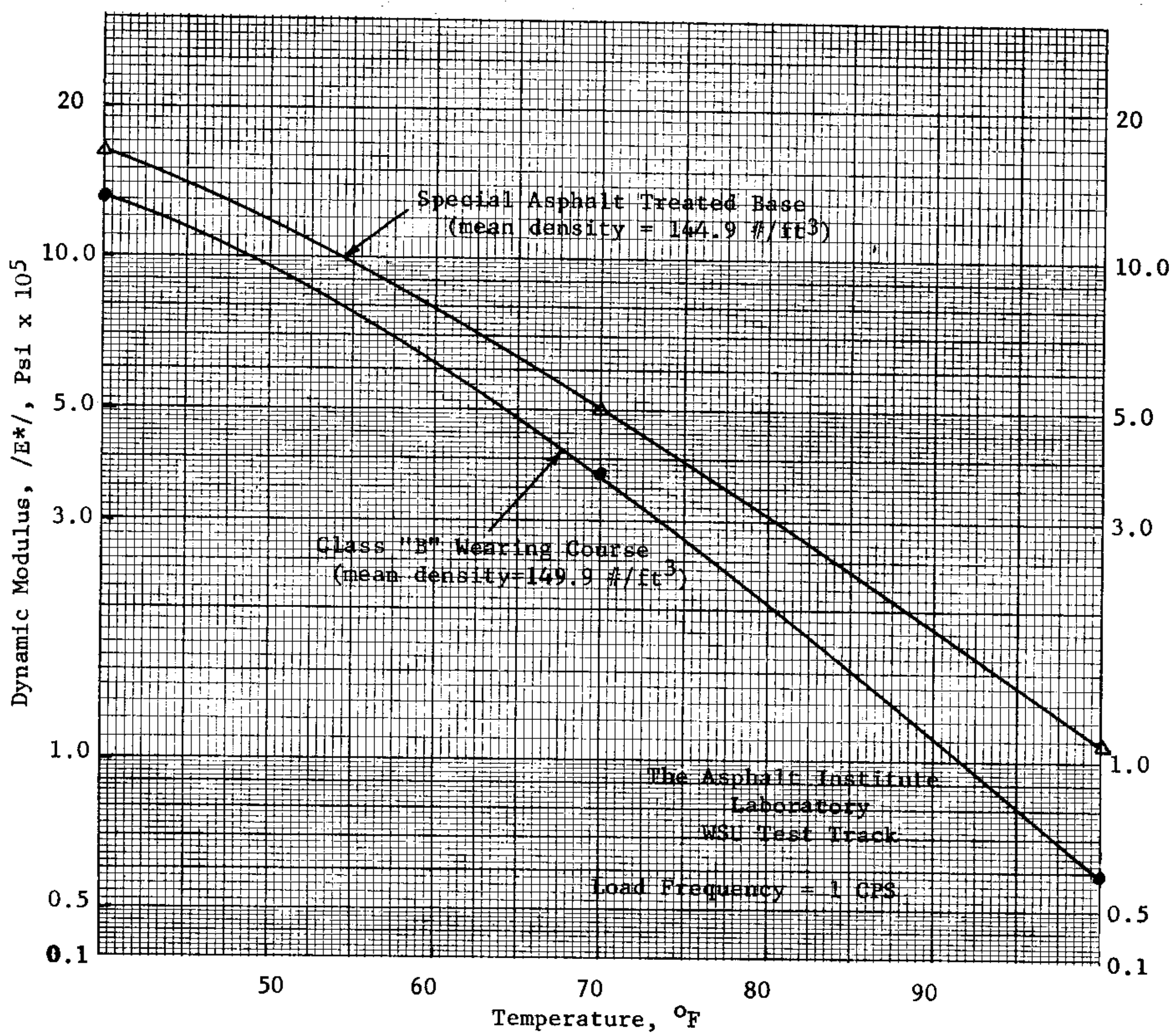


FIGURE 17
DYNAMIC MODULUS VS. TEMPERATURE RELATIONSHIPS FOR
ASPHALT TREATED BASE AND ASPHALT CONCRETE SURFACE COURSE



sections having the asphalt concrete base (ACB) were 6, 7 and 8 of Ring #4. Normal hot mix asphalt paving practices were followed (4).

Laboratory testing was similar to that for the asphalt treated base and the results are included in Figure 18.

Sand Asphalt Base

The sand came from a fine-grained, glacial lake deposit of clean, silica sand from northeastern Washington. This sand was hot-mixed with asphalt and laid as base in sections 1 to 4 in Ring #4. The road met all specifications (4). Normal hot mix asphalt paving practices were followed (4).

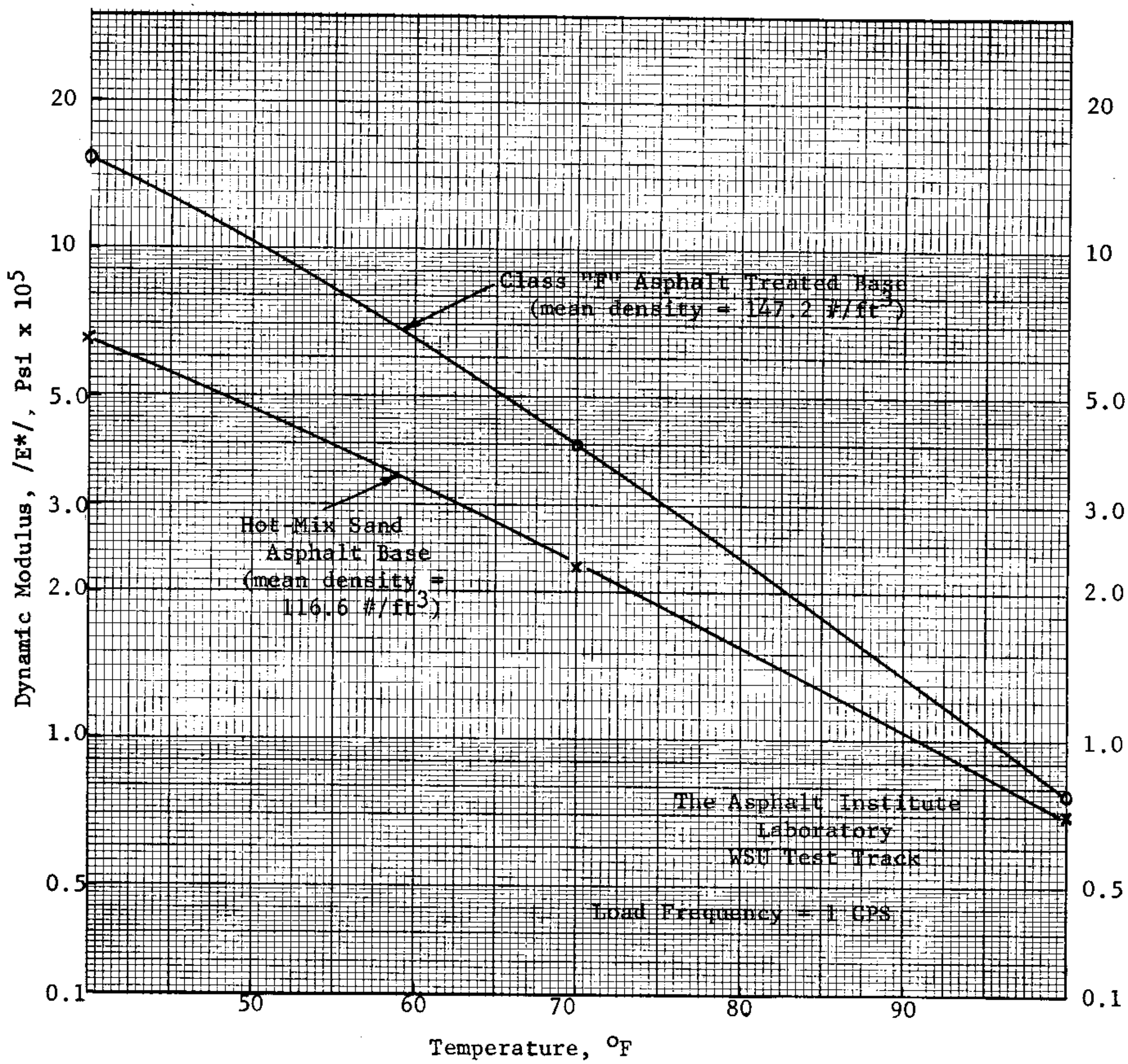
Laboratory testing was similar to that for asphalt treated base and the results are included in Figure 18.

Asphalt Concrete Surface

Crushed aggregate for the surface course was similar to that used in the untreated and emulsion treated bases, except for a somewhat different gradation; it met the Washington State Highway Department specifications for Class "B" material. All sections in Rings #2, #3 and #4 received a uniform three-inch thickness of this material and normal hot mix asphalt paving practices were followed (2, 3, 4).

Laboratory testing was similar to that for the asphalt treated base and results are included in Figure 17.

FIGURE 18
DYNAMIC MODULUS VS. TEMPERATURE RELATIONSHIPS FOR CLASS "F" ASPHALT TREATED BASE AND HOT-MIX SAND ASPHALT BASE



INITIAL EVALUATION OF FIELD DATA

Field data was provided (2, 3, 4) essentially in tabular form. Readings were made by either: (1) recording a complete set of readings from all instruments or (2) on a continuous basis. The former provided a look at the pavements' behavior with changing weather conditions such as temperature and rainfall as well as the effect of wheel load applications, while the latter readings helped to establish the response of instruments to lateral wheel position.

A primary objective of load tests is to compare the effects of repeated load applications for a range of pavement variables. The variables of interest in Rings #2, #3 and #4 were the following:

1. Base course, type (three in each ring)
2. Base course, thickness (four each per base type)
3. Number of wheel load repetitions to failure of each of the twelve test sections.
4. Behavior of pavements during application of repeated wheel loads (i.e., deflection, strain, pressure, etc.)
5. Environment (temperature of asphalt treated layers and water content of subgrade and some base types)
6. Lateral position of wheels with respect to instrumentation.

To effectively compare the relative merits of each type and thickness of pavement, several of the secondary variables must be reduced or eliminated. The effect of wheel position was used as an approximate means of observing the behavior of the pavements over their life span.

Fatigue Cracking

This method is approximate at best, but it is a way of observing the various relationships moving ahead with the analysis. The installation of strain gauges at the surface on all sections, and of several gauges at the bottom of the surface layer meant that the strain in the surface layer of each section could be reasonably determined.

The average "weighted" strain was estimated for the surface layer in each section. These were plotted with respect to the number of load applications at the time the first cracks appeared at the pavement surface for all three rings; this is shown in Figures 19, 20 and 21 for Rings #2, #3 and #4, respectively. These figures show that the points appear to be clustered rather closely where plotted on a log scale. Superimposed on these figures are data derived from fatigue tests (15) of asphalt concrete similar in nature to that in the test track. It is interesting to note that the field and laboratory data are in reasonable agreement. These comparisons illustrate that the test track field data are probably quite reasonable.

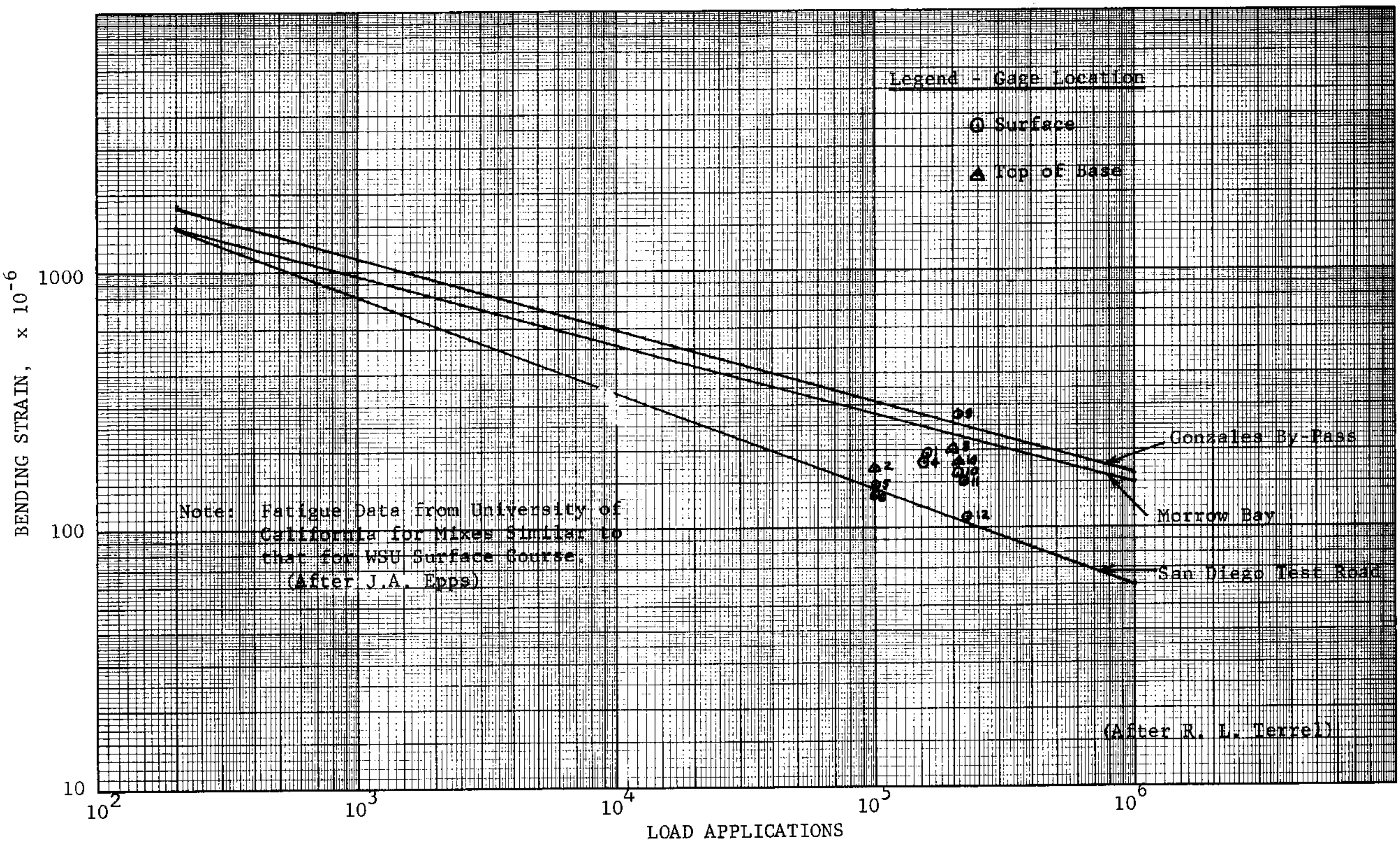


FIGURE 19

COMPARISON OF LABORATORY FATIGUE DATA AND WHEEL LOAD APPLICATIONS AVERAGE STRAIN AT INITIAL CRACKING OF SURFACE - RING #2

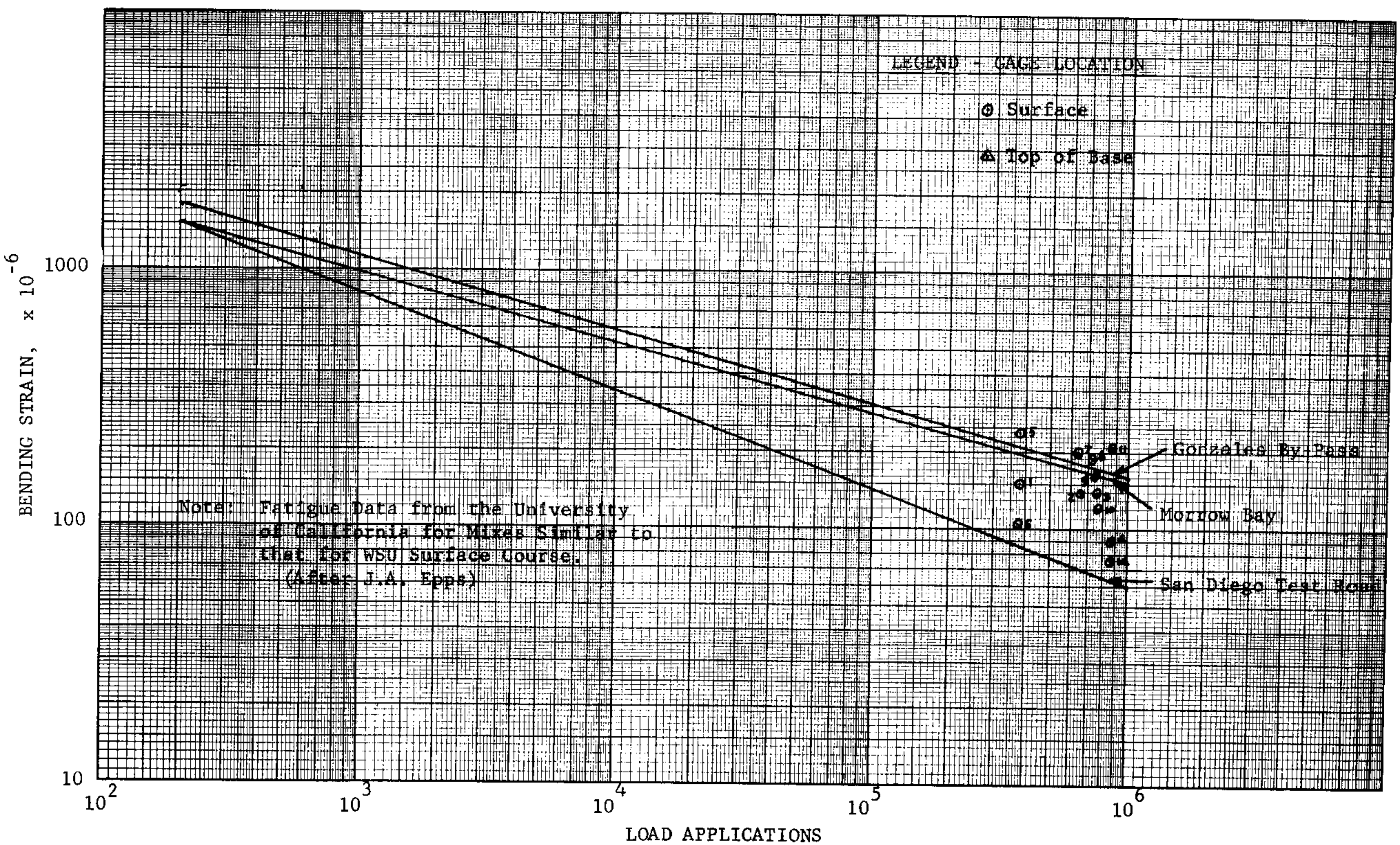


FIGURE 20
COMPARISON OF LABORATORY FATIGUE DATA AND WHEEL LOAD APPLICATIONS AVERAGE STRAIN AT INITIAL CRACKING OF SURFACE - RING #3

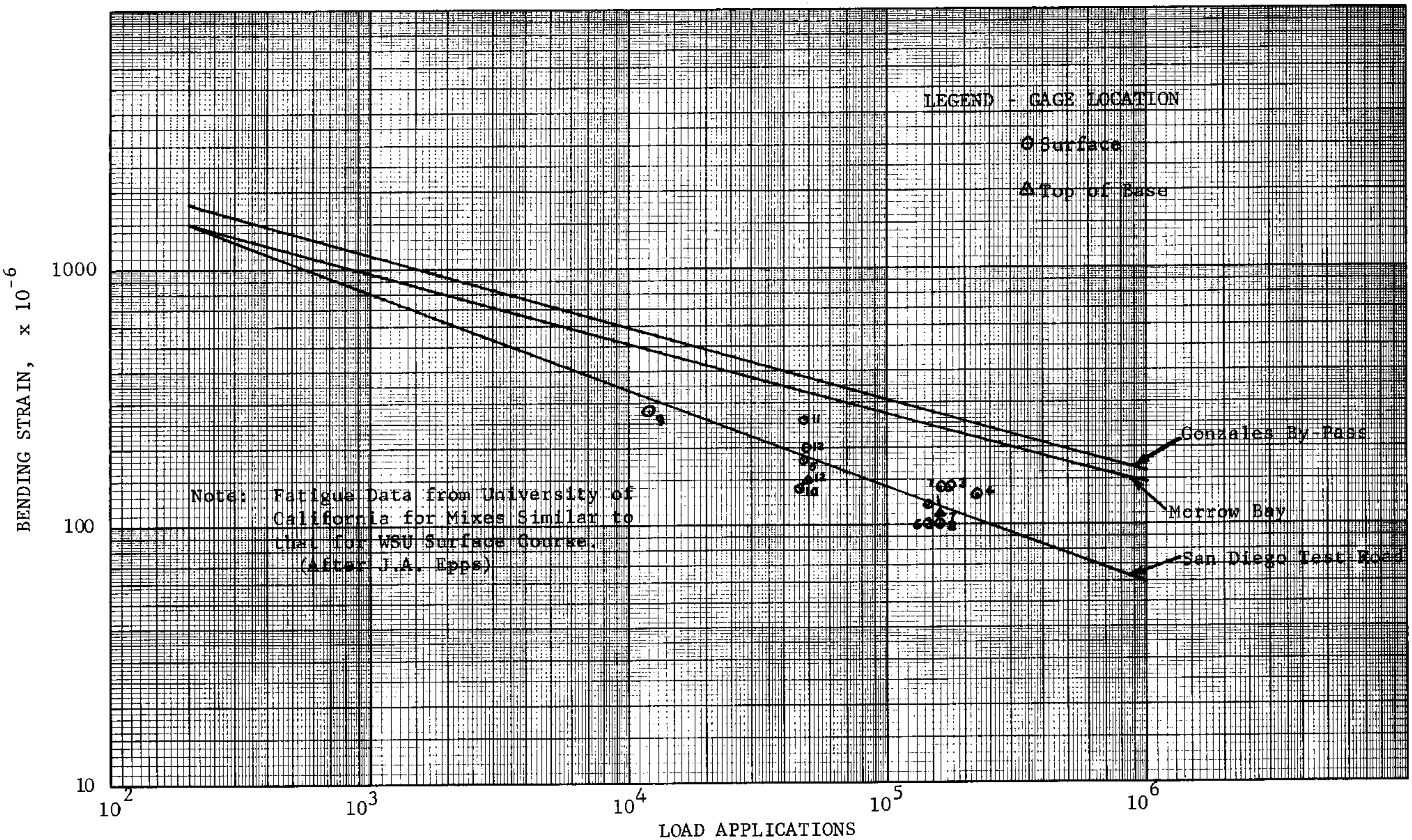


FIGURE 21.
COMPARISON OF LABORATORY FATIGUE DATA AND WHEEL LOAD APPLICATIONS AVERAGE STRAIN AT INITIAL CRACKING OF SURFACE - RING #4

TABLE 15: THE WHEEL LOAD APPLICATION RANGE USED IN THE N-LAYER ANALYSIS

Ring No.	Section No.	Number of Wheel Load Applications		
		Fall		Spring
		Before Rains	After Rains	
2 1966-67	2	48,000	100,000	187,000
	6	48,000	100,000	187,000
	8	48,000	100,000	208,600
	10	48,000	100,000	208,600
3 1967-68	3	280	339,000	736,000
	6	280	339,000	-------
	10	280	339,000	736,000
	12	280	339,000	736,000
4 1968-69	2	800	37,000	153,500
	4	800	37,000	153,500
	7	800	37,000	153,500
	10	800	37,000	-------

PREDICTION OF FIELD BEHAVIOR

The general nature of the measurements obtained during the time periods of testing for Rings #2, #3 and #4 indicates that pavement sections' behavior was different in the spring and fall of the year. This difference in pavement behavior necessitated the study of pavements' response to loading for a range of times. The instrumented pavement sections were studied in detail for specific numbers of wheel load applications as shown in Table 15.

The points in time, given in Table 15, were selected so as to represent conditions in the fall before the rains and cold weather began, (48,000, 280, and 800 wheel loads for Rings #2, #3, and #4, respectively), after a period of bad weather (100,000 339,000 and 37,000 wheel loads for Rings #2, #3 and #4, respectively), and in the spring after the winter shutdown and corresponding to a period of spring thaw. The general appearance and availability of data at these periods was considered. The data for the spring periods were usually sparse because of instrument breakdown; hence, computations were limited for this period.

To predict the potential for cracking or fatigue distress in the pavement test sections, one must first be able to predict the response of the pavement system to known loading conditions, using the laboratory test results and approximate theory.

Deflection obtained from LVDT and Benkelman Beam measurements offered the best results for comparison. Other types of measurements were also available and considered for comparison. Some selected measured data for the above sections are shown in the tables of Appendix D. The loading condition was constant at 10.6 kip dual tires. The position of the tires was also noted in tables of Appendix D for any particular measured value. Since the tires were 13 inches apart, a wheel position (noted in Appendix D, Table D-2, for example) of 2.9 + 10.1 indicates that one time centerline was located 2.9 inches from the track centerline (also location of instruments) and the other 10.1 inches.

Material Properties

The properties of these materials utilized in this study were previously presented in an earlier section. Briefly, the data utilized are as follows:

1.	Subgrade Soil	Figures 11 and 12
2.	Untreated Aggregate Base	Figure 14
3.	Emulsion Treated Base	Figure 16
4.	Asphalt Treated Base	Figure 17
5.	Asphalt Concrete Base	Figure 18
6.	Sand Asphalt Base	Figure 18
7.	Asphalt Concrete Surface	Figure 17

In addition to the stress-deformation characteristics represented in the above data, information regarding Poisson's ratio for each of the pavement materials in the section is necessary. The following values were estimated as being suitable for the materials used:

Subgrade soil - 0.35

All other materials - 0.40

Deflection and Stress Calculations

The calculation of stresses and deflections within a pavement depends upon the assumption that the section behaves as a semi-infinite layered system. This permitted the utilization of the computer solutions for such a system developed by the Chevron Research Corporation (16). This program provides the ability to compute the magnitudes of stresses and deflections and/or strains at any point within the structural section under the application of a single circular load with a uniformly distributed pressure. The program will accommodate up to 15 layers of different material, but a maximum of 8 were utilized in this analysis. Some modifications had to be made so that this program was compatible with the W.S.U. computer system.

The wheel load, for calculation purposes, was assumed to be carried on two circular areas with a contact pressure equal to 80 psi in each tire. Since the moduli of the material in the pavement layers (particularly the untreated and emulsion treated bases) are dependent on the stress levels existing in the strata, an iterative procedure is required to insure compatibility between the modulus and the stress state in any layer. Since, as noted above, the computer solution affords the means of determining the stresses and displacements at any point in the loaded system, the effect of the dual load on any point is then determined by linear superposition of the effects of each of the loads at the point in question. This application of superimposition implies linear response which makes the utilization of the principle somewhat of an approximation.

Seed, et al (17), an early user of the n-layer concept of pavement analysis, utilized an interative procedure for a single loaded area. The use of the dual wheel simulation necessitates several changes in the iterative method since the simulative no longer represents an axisymmetric load condition. These changes were being incorporated in this analysis at about the time Monosmith, et al, (18) reported using the same concept, thus the following description essentially parallels their procedure. The most important change from the former method is caused by the fact that away from the axis of the load, the radial and tangential stresses are no longer equal. Figure 22 shows the relationship of these stresses. This difference makes impossible the selection of a confining pressure for determination of the appropriate modulus of a granular or emulsion treated material. This is the reason why the relationship between the resilient modulus and the sum of the principal stresses (bulk stress) is used.

The iterative method employed in this analysis has been previously used by Terrel (11) and is outlined below:

Iterative Method for Dual Tire Loading

1. The modulus of each layer is estimated from laboratory data and field conditions noted at the time under consideration.
2. Using these moduli, the stresses are calculated at several depths within each layer and at radial distances from the axis of a loaded area convenient for superposition and corresponding to the lateral wheel position at the time under consideration.
3. The sum of principal stresses caused by the load is determined at each point within the layered system by superposition of the sum of stresses caused by each of the loaded areas. For the solutions

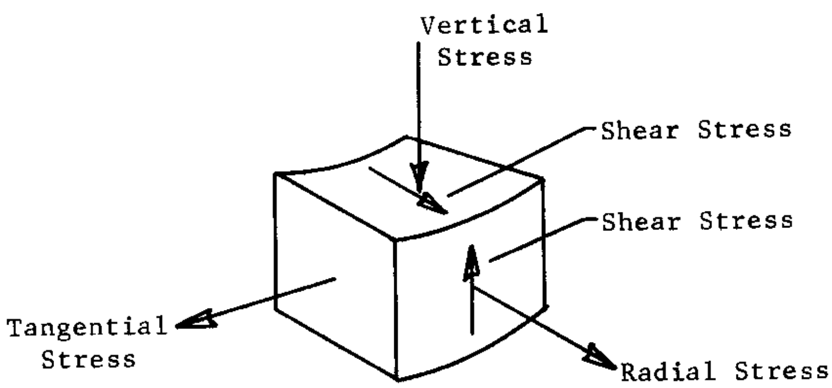


FIGURE 22
STRESSES ON AN ELEMENT R INCHES FROM THE AXIS
OF THE LOADED AREA AND Z INCHES BELOW THE SURFACE.
After Reference (18)

discussed herein, the two circular areas were at a spacing of 13 inches.

4. Generally, the sum of principal stresses due to overburden should be considered, but in the initial phases of this analysis they were omitted because the pressure gauges were adjusted to zero reading when no wheel load was present.
5. The modulus at each point in depth and horizontal points corresponding to the wheel positions is determined from the appropriate modulus vs. bulk stress relationship.
6. The mean value of the modulus for each layer is thus determined and the modulus values compared with those estimated at the outset.
7. The process is repeated until compatibility is obtained between the stresses and the moduli for the materials.

The steps outlined above cam be programmed for computer solution (18), but for this analysis a slightly different approach was used. The basic Chevron program was altered to give tangential strain rather than radial strain. In addition, the results can be in the form of punched cards. The superimposition of dual (or multiple) wheels can then be accomplished and summarized by computer. For the case used here, with the dual tires 13 inches apart, if the position of one tire is known, the results will show the sum of the effects of both tires. A further useful method of presenting the computed values was utilized. Individual parameters such as deflection, vertical stress, etc., can be plotted vs. lateral distance from the center of loaded area. No examples are included.

DISCUSSION OF RESULTS

The tables in Appendix D show typical results obtained using this method for the heavily instrumented sections in the three rings, and also include the distribution of bulk stress with depth below the dual wheels (maximum values). Computations for times representing several different wheel load applications were made, but only wheel loads for 100,000, 339,000 and 37,000 for Rings #2, #3 and #4, respectively, are shown here. The reasons for selecting these particular wheel load applications for the fall are that in all three rings cracks were observed soon afterwards, and most of the instruments were still functioning. The tables in Appendix D also show that the base course was subdivided into two or more layers, especially for the ETB and UTB. This was done to prevent the tendency for tensile stresses to be computed in these layers which is a common problem in this type of analysis.

The computed total (deep) deflection values which measured the total pavement system rarely matched those measured in most instances. The probable explanation for serious difference upon study of the data is questionable field measurements. For Ring #2, Benkelman beam readings were substantially lower than predicted due to a problem in the method of measuring these deflections. Benkelman beam measurements in Rings #3 and #4 were found to be correct and complimented the dynamic deflection readings and were of great help in matching computed values.

Shallow deflection (in surface and base courses which constituted the pavement structure) measured in the pavement were generally somewhat higher than those computed and appear to be erratic. This problem is perhaps attributable to the great sensitivity of these shallow gages which caused difficulty in measuring small values accurately over a long period of time.

Both lateral and longitudinal strains were measured in the pavements. The latter were selected for analysis because it appeared that they may be somewhat easier to predict. Results show that even these were difficult to predict, probably because of the sensitivity of strain gages to small changes in the position of wheels. Despite these difficulties, many computed values were quite comparable to those measured at the surface and in the base in some instances. Measured strains in the subgrade were consistently too low, which may be due to the nature of the strain mounting technique at these positions on the pavement.

Vertical pressure, where measured using the WSU hydraulic pressure cell, was consistently lower than that computed. The response of these instruments may not be rapid enough for the transient loads applied. Pressure measured with the Filpip devices showed considerable scatter as can be observed from the data. The WSU strain gage cell seemed to give consistent results, although on the lower side as compared with computed results.

It can be summarized that the prediction of deflections (measured using LVDT's and Benkelman beam rebound readings) using layer elastic theory along with material properties determined from dynamic tests on laboratory prepared samples is reasonably good. The use of thermocouples placed at several levels in the pavement system helped to determine the proper material property values for particular ranges in laboratory conditions. Even so, a serious deficiency in the method used, however, is that the material properties used in this analysis were those determined for particular ranges in laboratory conditions. Despite the accurate determination of modulus values, it is difficult to assign appropriate values to these materials in the pavement at various times during the pavement's life. Field measurements should be accompanied at the same time by sampling and testing of the pavement materials which would make it substantially easier to establish the effectiveness of this type of analysis and the validity of elastic layer theory in general. Some sampling and testing of pavement materials in Ring #4 was done in conjunction with field measurements and the results were generally favorable.

LAYER EQUIVALENCIES OF DIFFERENT BASE TYPES

Equivalency ratios to the various base materials can be assigned by various ways; namely, theoretical and field observations. Equivalencies obtained by field observations have been previously mentioned in an earlier chapter. Equivalencies obtained by theoretical measurements have been taken with some reservations due to the general nature and limitations of these experiments. One approach will be attempted to illustrate at least one method of assigning equivalencies.

Several points in time during the course of field testing were selected for analysis. Only one point in time was selected for equivalency comparisons because more information was available and cracks started to appear in the fall testing period soon afterwards. These points in time were 100,000 wheel load computations for Ring #2, 339,000 wheel load computations for Ring #3 and 37,000 wheel load computations for Ring #4. It certainly would be more meaningful if all times could be considered and properly weighed to account for environmental changes as well as changing material properties such as curing of asphalt emulsions.

Although various criteria have been suggested for measuring pavement performance, three have been selected as being representative:

1. Deflection
2. Vertical stress on the subgrade, and
3. Strain in the asphalt layers.

Utilizing the information regarding pavement behavior and material properties for pavement sections 2, 6, 8 and 10 at 100,000 wheel load applications for

Ring #2, pavement sections 3, 6, 10 and 12 at 339,000 wheel load applications for Ring #3, and pavement sections 2, 4, 7 and 10 at 37,000 wheel load applications for Ring #4, similar computations were made for all twelve sections. This made it possible to observe the above parameters as the thickness of the base course changed. In most cases, modulus values for the surface and subgrade were held constant. The monitoring of pavement temperatures in Rings #3 and #4 allowed the accurate determination of surface modulus. It was found that a boundary condition existed in Rings #3 and #4 at section 6 where part of the section was by the concrete tunnel. Here a lower subgrade modulus was established which may be due to the trapping of moisture by the concrete structure. It was found in Ring #4 that the subgrade modulus varied from section to section, especially under the untreated bases, and is probably due to the unfavorable environmental conditions which existed during construction and testing. Figures 23 to 31 show the results of these computations, which were accomplished using a technique similar to that described earlier.

Assigned values to the above criteria for extremes of moisture and temperature during summer and winter would make the results more meaningful; only the following were used for the comparison:

Deflection		0.025 inches
Stress on Subgrade		7.0 psi
Strain in Bottom of;	ATB, ACB	150×10^{-6} in./in.
	Surface	300×10^{-6} in./in.

These values are all somewhat higher than might be normally used for design purposes, but appear to fit within the limits of the experiment. For all practical purposes, the ATB is quite similar to the ACB. The equivalencies for the different rings resulting from these considerations are shown below in Table 16 for Ring #2, Table 17 for Ring #4 and Table 18 for Ring #4.

TABLE 16

LAYER EQUIVALENCY VALUES BASED ON PAVEMENT CONDITION
AFTER 100,000 WHEEL LOAD REPETITIONS IN TERMS OF ASPHALT TREATED BASE

RING #2

(After R. L. Terrel)

Base Material	Deflection	Stress	Strain Base	Field*
Asphalt Treated	1.00	1.00	1.00	1.00
Emulsion Treated	∞ **	6.1	5.8	3.50
Untreated	∞ **	5.5	9.2	4.75

*From Table XXV, Ref. (2), for Fall, 1966.

The above table and Figures 28-30 indicate the difficulty of assigning precise values for all situations. For example, the total deflection of those pavements with ETB and UTB was always above 0.030 inches and appears to stay relatively constant over the full range of thicknesses. Strain in the asphalt layers also was difficult to select. Tensile strain in the bottom of the asphalt treated changed significantly with base thickness, but strain in the bottom of the 3-inch asphalt concrete surface course was relatively constant over the range of base thicknesses. Even though the emulsion treated base might ultimately be able to resist tensile stress or strain, at the time of these comparisons, it was essentially uncured and consequently was considered to be similar to untreated aggregate.

**No extrapolation of the deflection data from Figure 23 could be obtained so that for all practical purposes the relationship of base thickness to deflection can be said to be infinite.

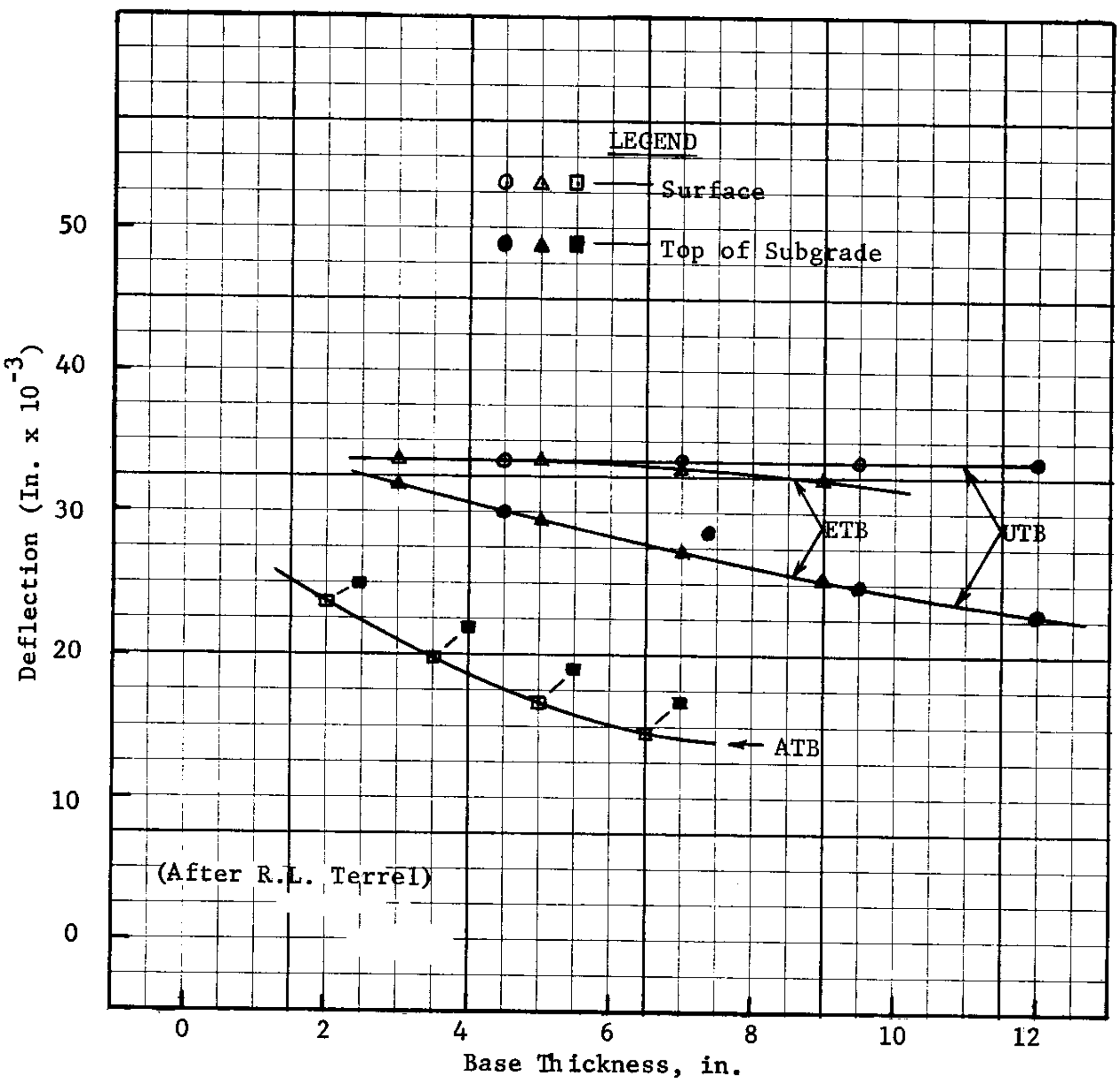


FIGURE 23
COMPUTED DEFLECTION VS. BASE THICKNESS RELATIONSHIP
Ring #2 - 100,000 wheel loads

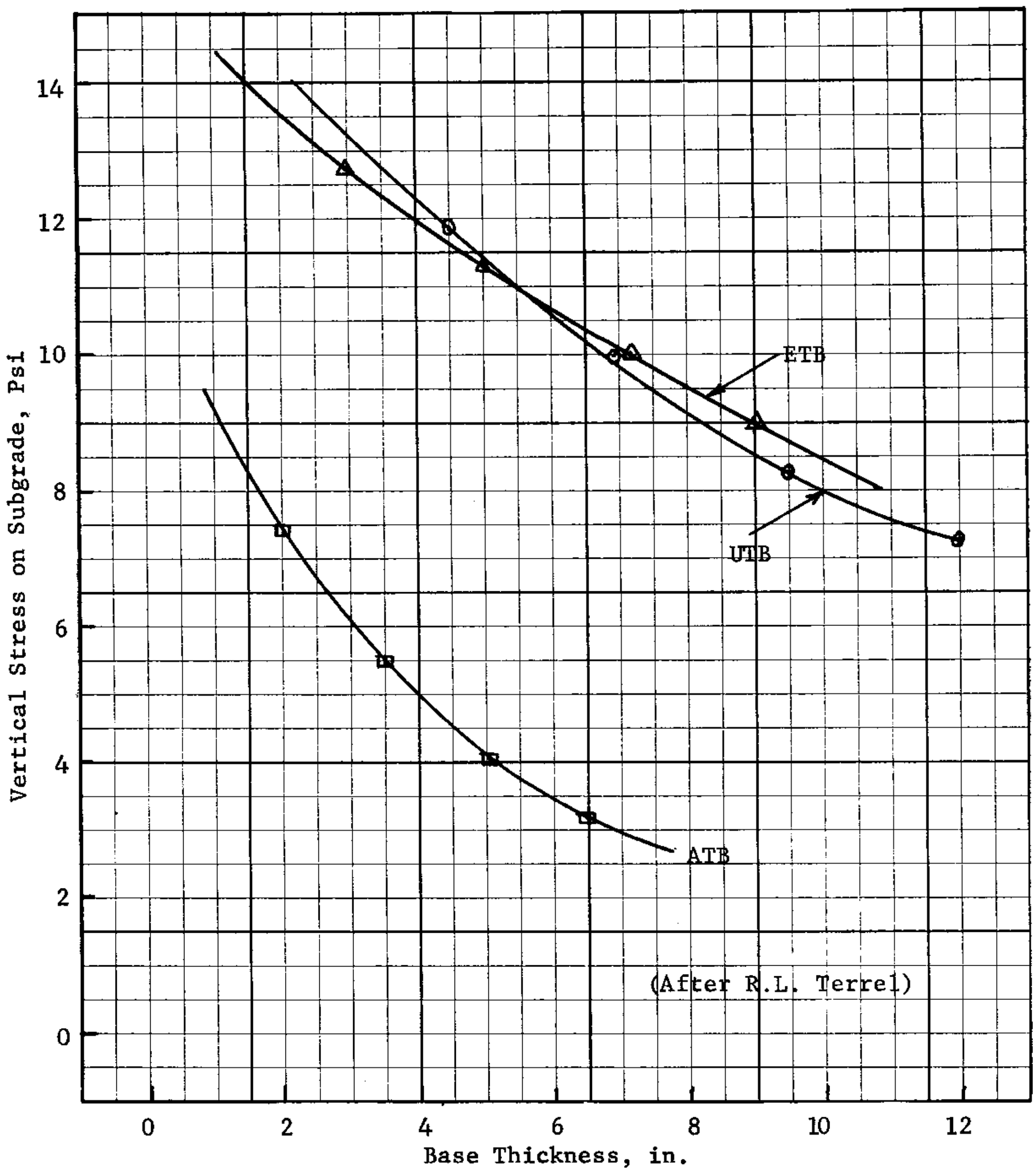


FIGURE 24
COMPUTED VERTICAL STRESS ON SUBGRADE VS. BASE THICKNESS RELATIONSHIP
Ring #2 - 100,000 wheel loads

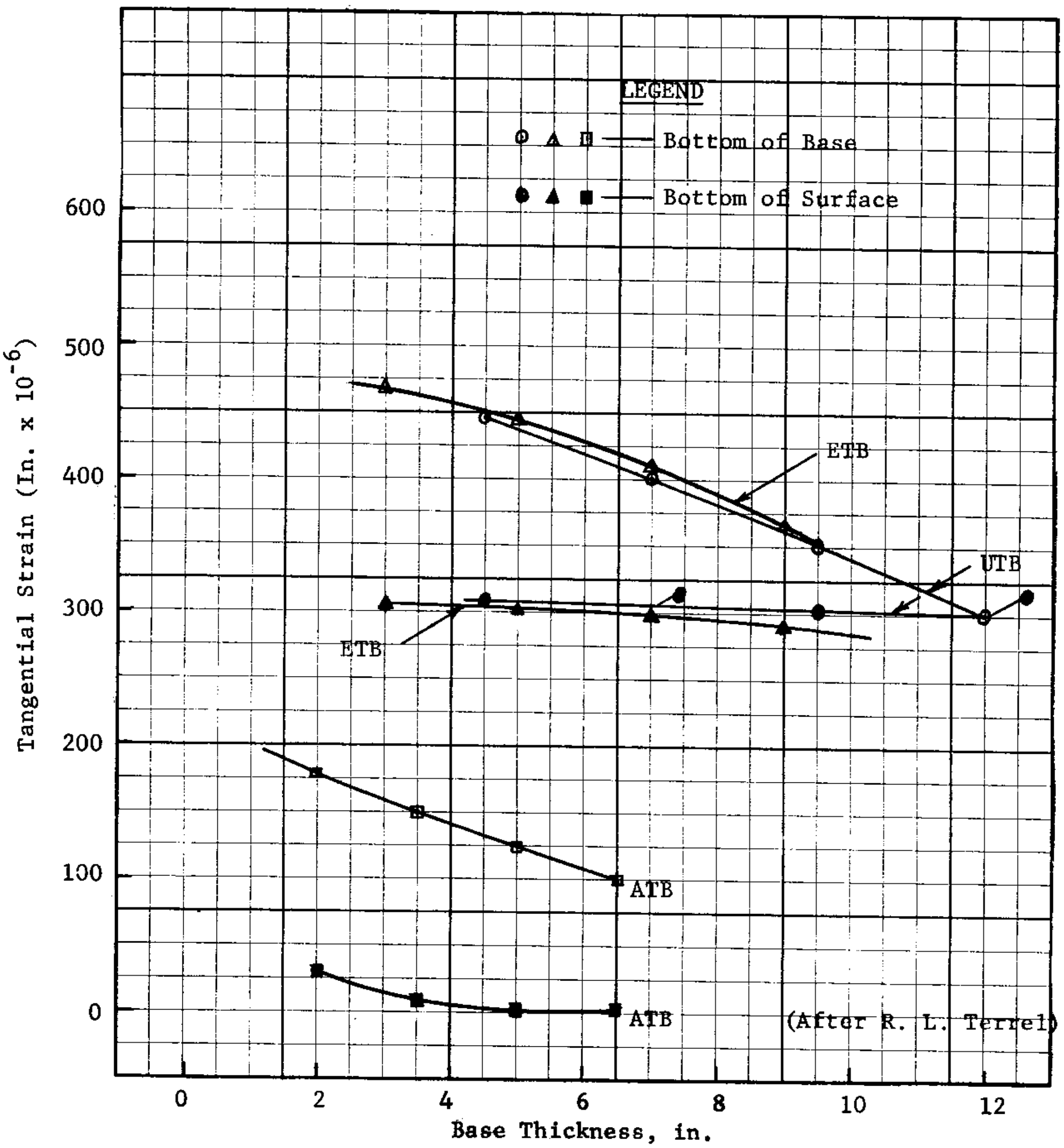


FIGURE 25
COMPUTED TANGENTIAL STRAIN VS. BASE THICKNESS RELATIONSHIP
Ring #2 - 100,000 wheel loads

TABLE 17

LAYER EQUIVALENCY VALUES BASED ON PAVEMENT CONDITION
AFTER 339,000 WHEEL LOAD REPETITIONS IN TERMS OF ASPHALT TREATED BASE

RING #3

Base Material	Deflection	Stress	Strain - Bottom of		Field*
			Surface	Base	
Asphalt Treated	1.00	1.00	1.00	1.00	1.00
Emulsion Treated	2.85	3.44	9.20	9.00	1.50
Untreated	5.83	4.70	22.00	24.00	2.75

*From Table 29, Ref.(3), for Fall 1967.

Figures 26-28 and the above table show how difficult it is to assign precise values. From Figure 26, one can see that some total deflections for the ATB and ETB were under 0.025 inches, while for the UTB, they were above this level. By extending the graph, one was able to obtain a deflection value for the UTB. No problems were found in selecting equivalencies from the stress values shown in Figure 27. Tensile strain at the bottom of the ATB and ETB changed significantly with base thickness. Strain in the bottom of the 3-inch asphalt concrete surface was not relatively constant as found in Ring #2. The UTB had significantly higher tensile strains than either the ATB or ETB which shows up in the equivalency figures. In Ring #3, the ETB was found to be superior than that in Ring #2. This was probably due to the fact that the ETB was relatively cured as compared to the uncured condition in Ring #2. The field equivalencies seem to be low compared to the computed theoretical values.

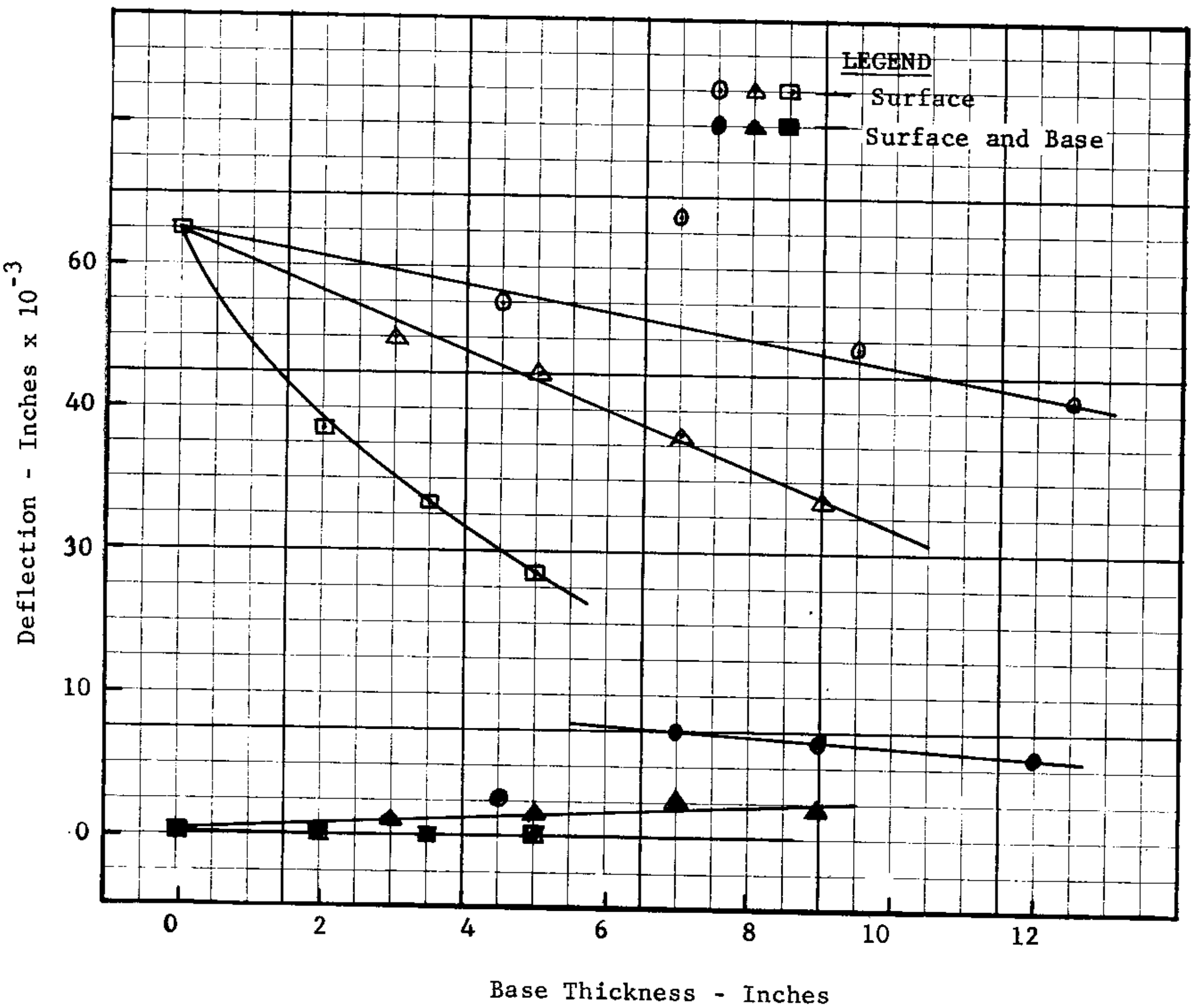


FIGURE 26
COMPUTED DEFLECTION VS. BASE THICKNESS RELATIONSHIP
Wheel Load = 339,000 - Ring #3

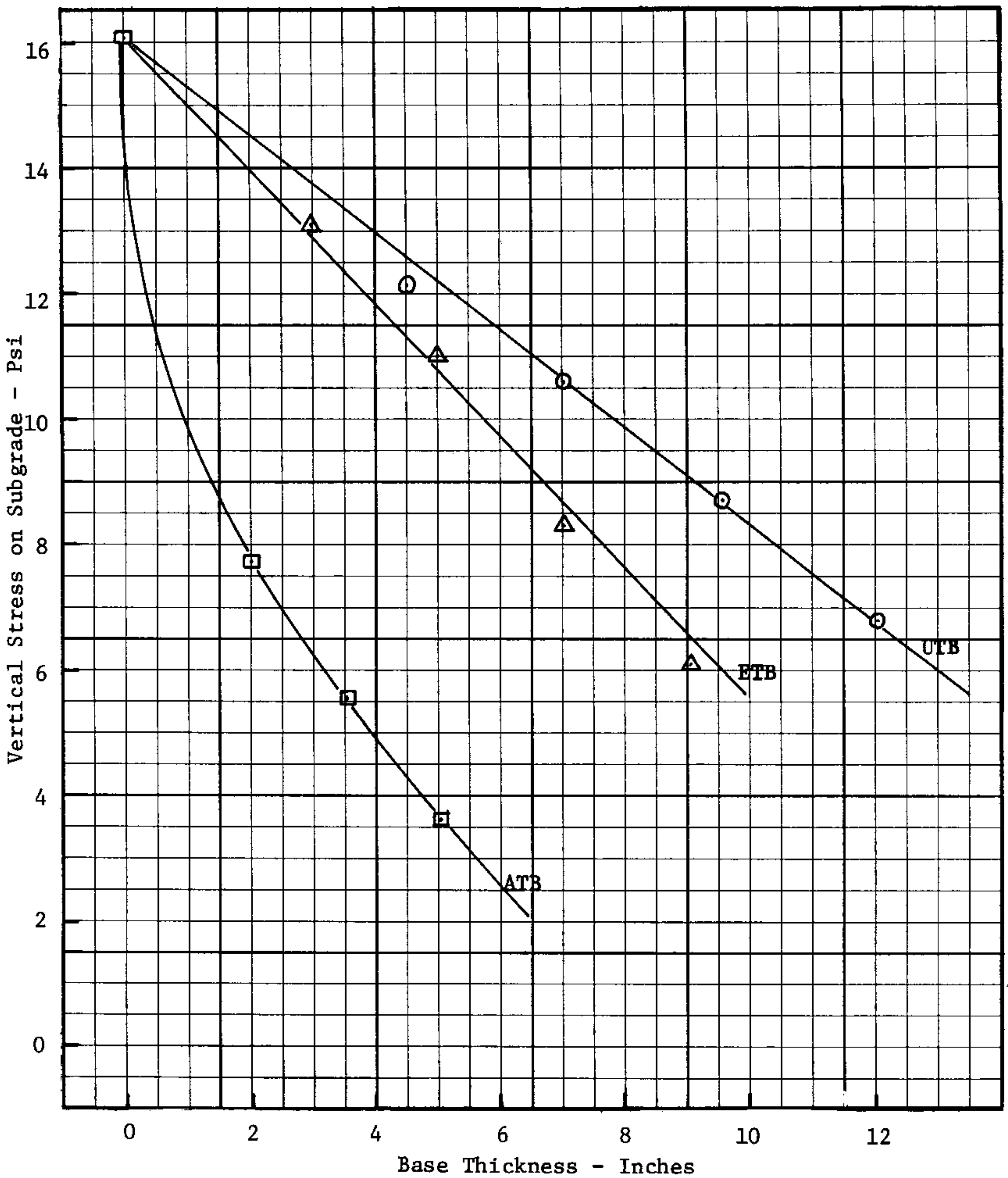


FIGURE 27
COMPUTED VERTICAL STRESS ON SUBGRADE VS. BASE THICKNESS RELATIONSHIP
WHEEL LOAD = 339,000 - RING #3

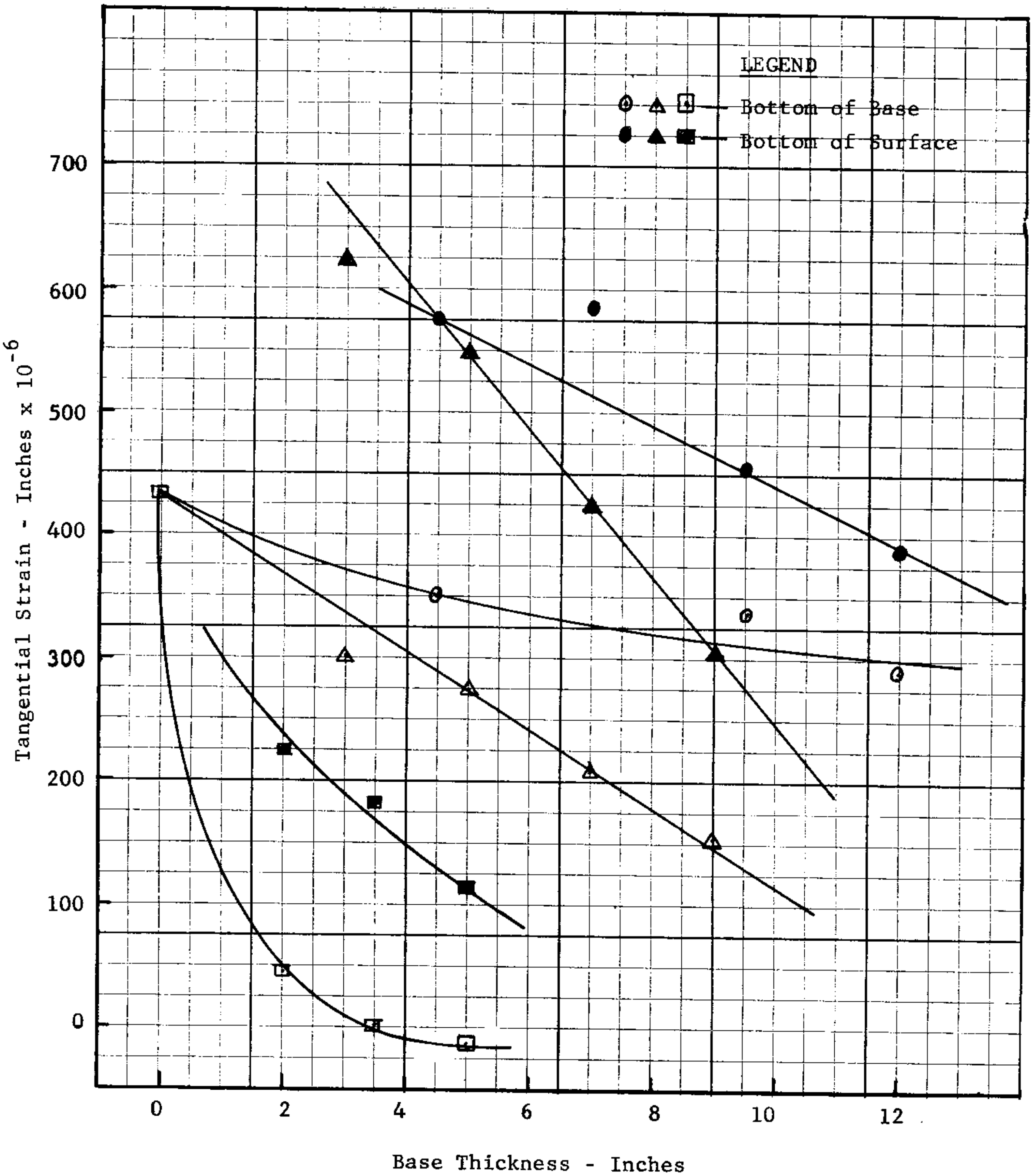


FIGURE 28
COMPUTED TANGENTIAL STRAIN VS. BASE THICKNESS RELATIONSHIP
Wheel Load = 339,000 - Ring #3

TABLE 18

LAYER EQUIVALENCY VALUES BASED ON PAVEMENT CONDITION
AFTER 37,000 WHEEL LOAD REPETITIONS IN TERMS OF ASPHALT CONCRETE BASE

RING #4

Base Material	Deflection	Stress	Strain - Bottom of		Field*
			Surface	Base	
Asphalt Concrete	1.00	1.00	1.00	1.00	1.00
Sand Asphalt	0.92	1.30	1.00	1.25	1.00
Untreated	8.85	4.10		6.06	4.75

*From Table 22, Ref.(4), for Fall 1968.

Figures 29-31 and the above table indicate the difficulty of assigning precise values to all situations. The total deflections shown in Figure 29 show that both the ACB and SAB were similar and there was little to choose between them. The UTB deflections were very high. Figure 30 shows there was a little difference between the ACB and SAB for stress levels. The UTB had the highest stress levels. Figure 31 shows the tensile strains developed at the bottom of the surface and bases for the different materials. The UTB had the highest strain while the SAB and ACB strains were similar and lower than the former. The equivalencies developed show the similarity in strength of the ACB and SAB at this wheel load repetition. This compares favorably to the field equivalencies. The figures indicate that the UTB conditions, probably both the subgrade and base, were somewhat different than those found under the ACB and SAB.

**The relationship of base thickness to strains obtained by extrapolation from Figure 31 was 125.00 miles, which is so high that for practical purposes it can be said to be infinite.

The theoretical equivalencies which compared best to the field equivalencies were those developed from the stress conditions. The least comparable equivalencies were the ones developed from the strains. This was true for all rings. In all rings, the UTB developed the highest values of deflection, strain and stress in comparison to the different treated bases.

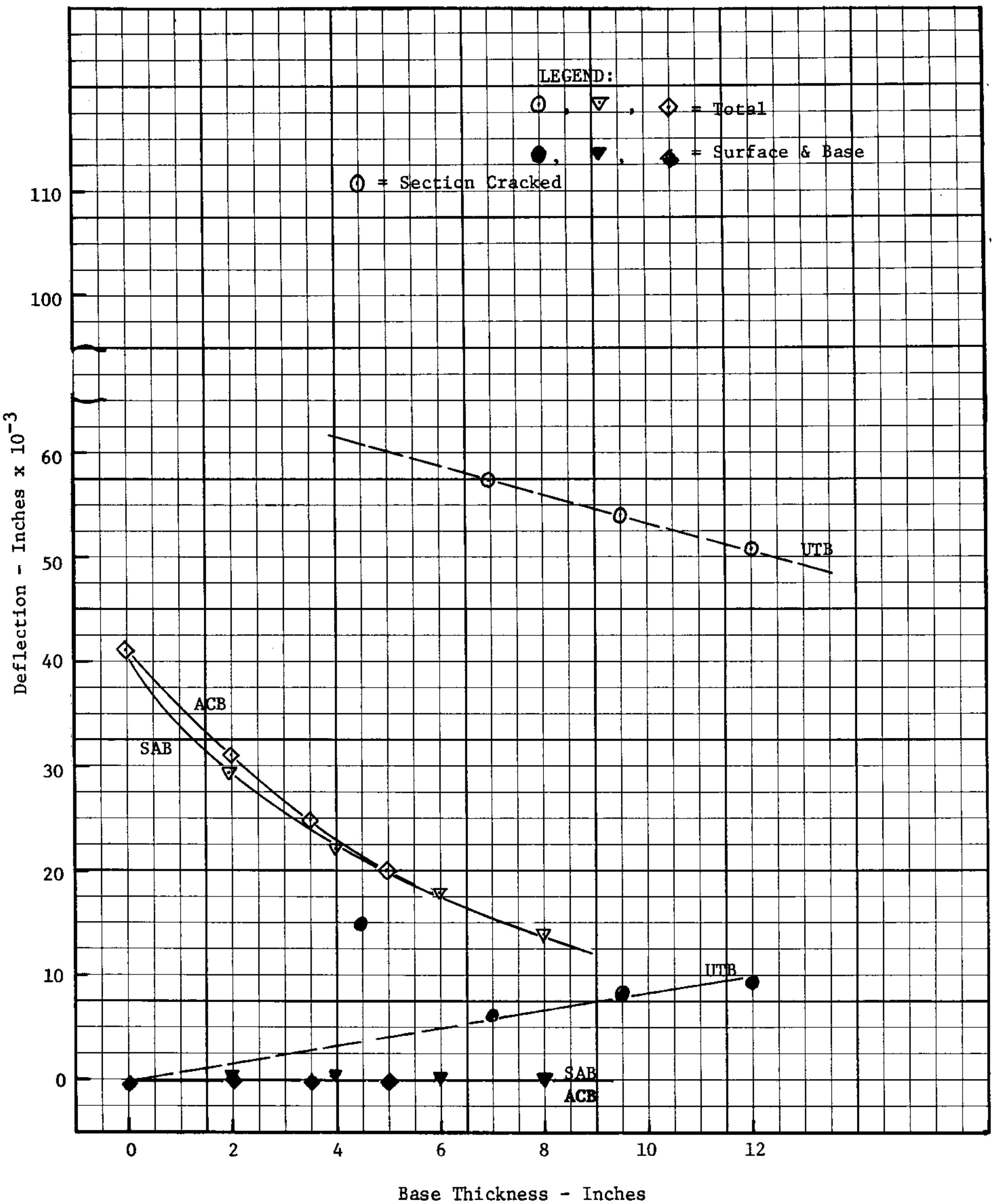


FIGURE 29

COMPUTED DEFLECTION VS. BASE THICKNESS RELATIONSHIP WL=37,000 - RING #4

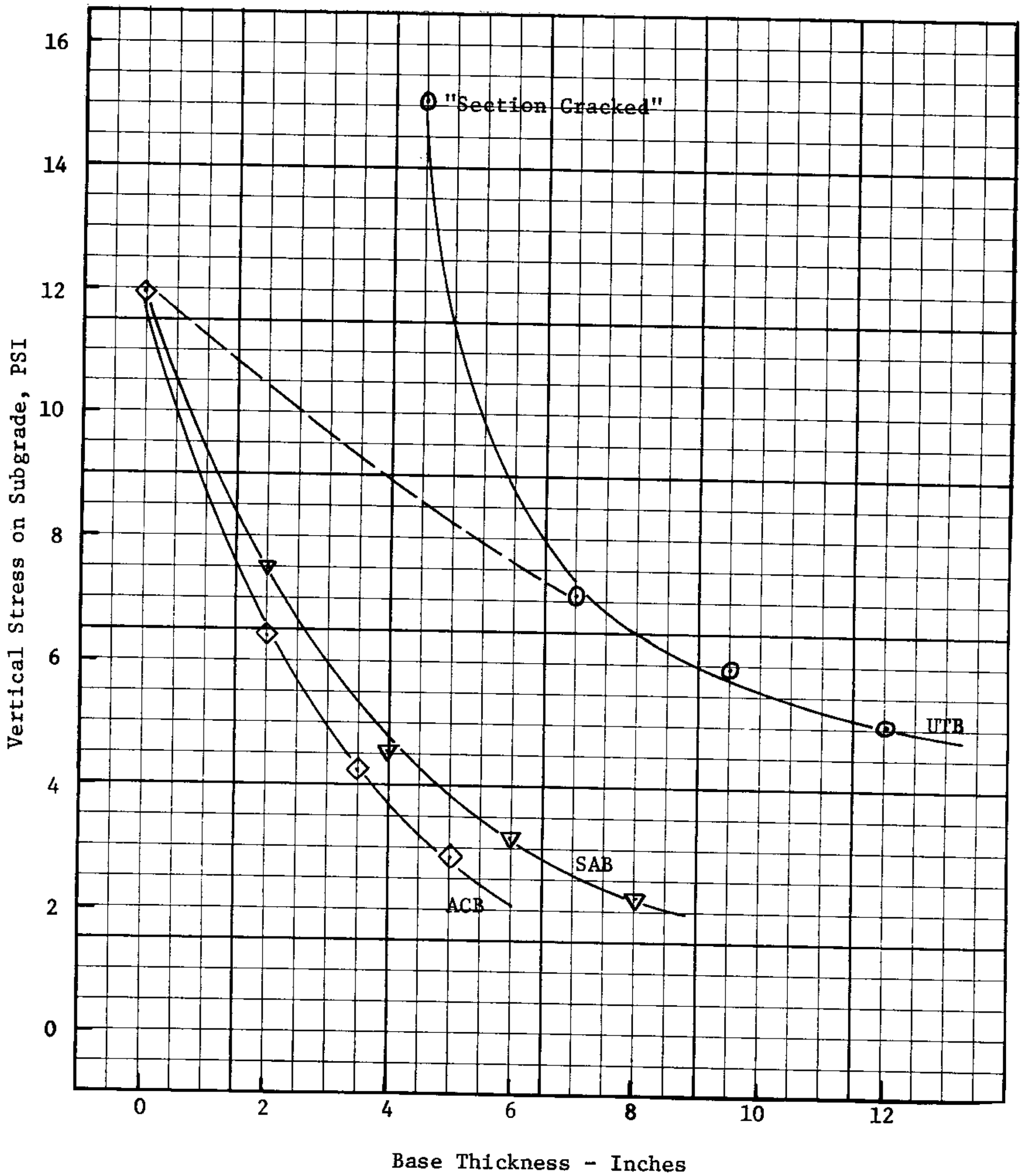


FIGURE 30
COMPUTED VERTICAL STRESS ON SUBGRADE VS. BASE THICKNESS RELATIONSHIP
WL = 37,000 - RING #4

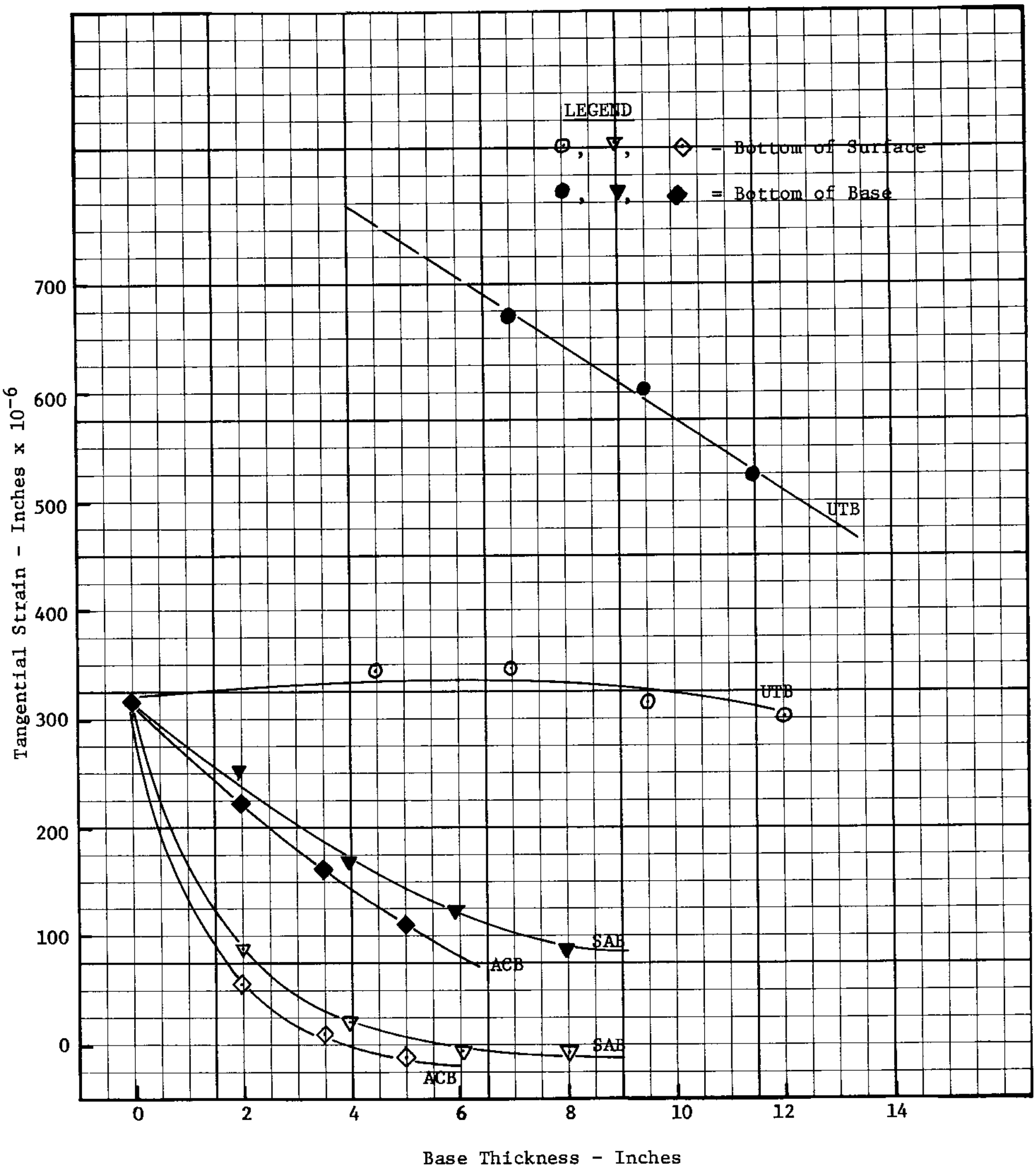


FIGURE 31
COMPUTED TANGENTIAL STRAIN VS. BASE THICKNESS RELATIONSHIP
WL = 37,000 - RING #4

CONCLUSIONS AND RECOMMENDATIONS

The test ring and apparatus appeared to be suitable for comparison of different pavement materials.

The theoretical results computed from n-layer elastic theory indicated that the measure pavement response was reasonable, especially for deflections and stresses, but not so well for strains. The scatter or inconsistency of data may be a function of either location of dual wheels at the time of measurements or of poorly functioning instrumentation. The general trends over the length of the experiment are favorable.

Despite having functioning thermocouples in Rings #3 and #4, it is still difficult to define changes and behavior in base and subgrade conditions with time. A reliable instrument for monitoring moisture content changes is definitely needed. A combination of better instrumentation and continued sampling would be desirable in similar future work.

The results from these tests appeared to be reasonable theoretically. This should allow the development of critical stress, strain and deflection values for design purposes. This work is being done in the continuing study of data.

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APPENDIX A

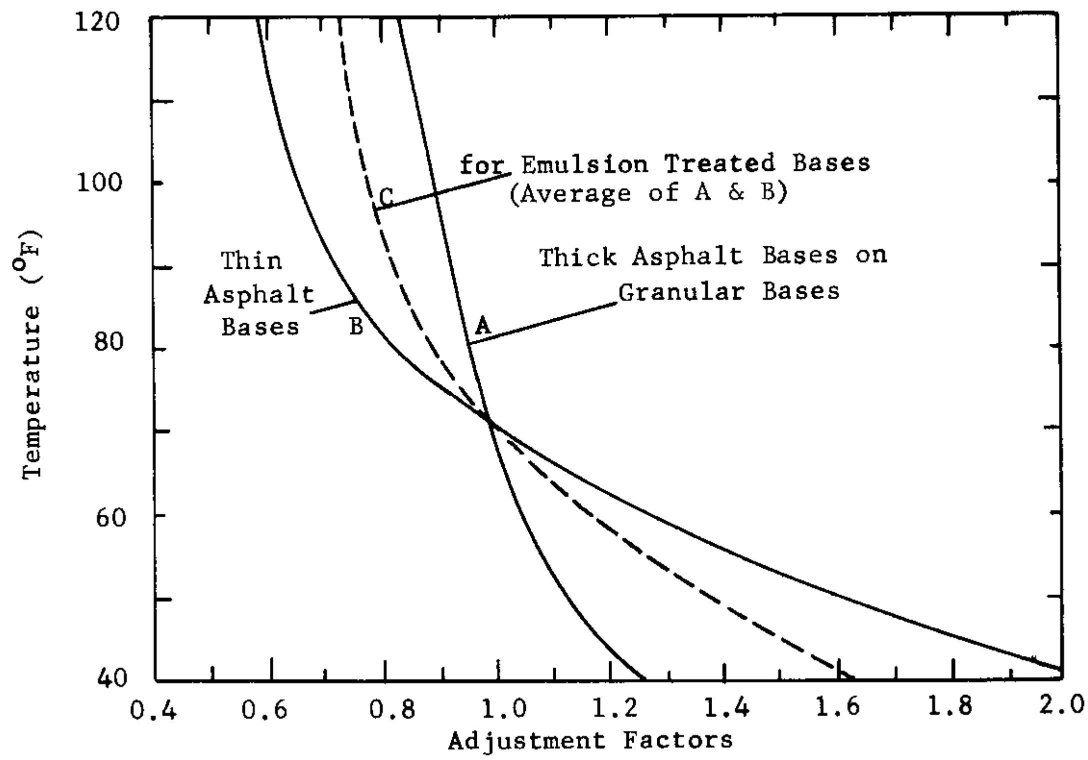


FIGURE A-1
ADJUSTMENT FACTORS FOR BENKELMAN BEAM DEFLECTIONS AT VARIOUS TEMPERATURES.

Note: "A" above is the curve for granular base pavements with thick asphalt bases. "B" is the curve for pavements with four inches or more of total asphalt thickness on a weak foundation. "C" is the adjustment factor for the emulsion treated bases at various temperatures, and is the average of A & B.

Source: R. Ian Kingham, "A New Temperature Correction Procedure for Benkelman Beam Rebound Deflections," The Asphalt Institute, Research Report 69-1, February 1969.

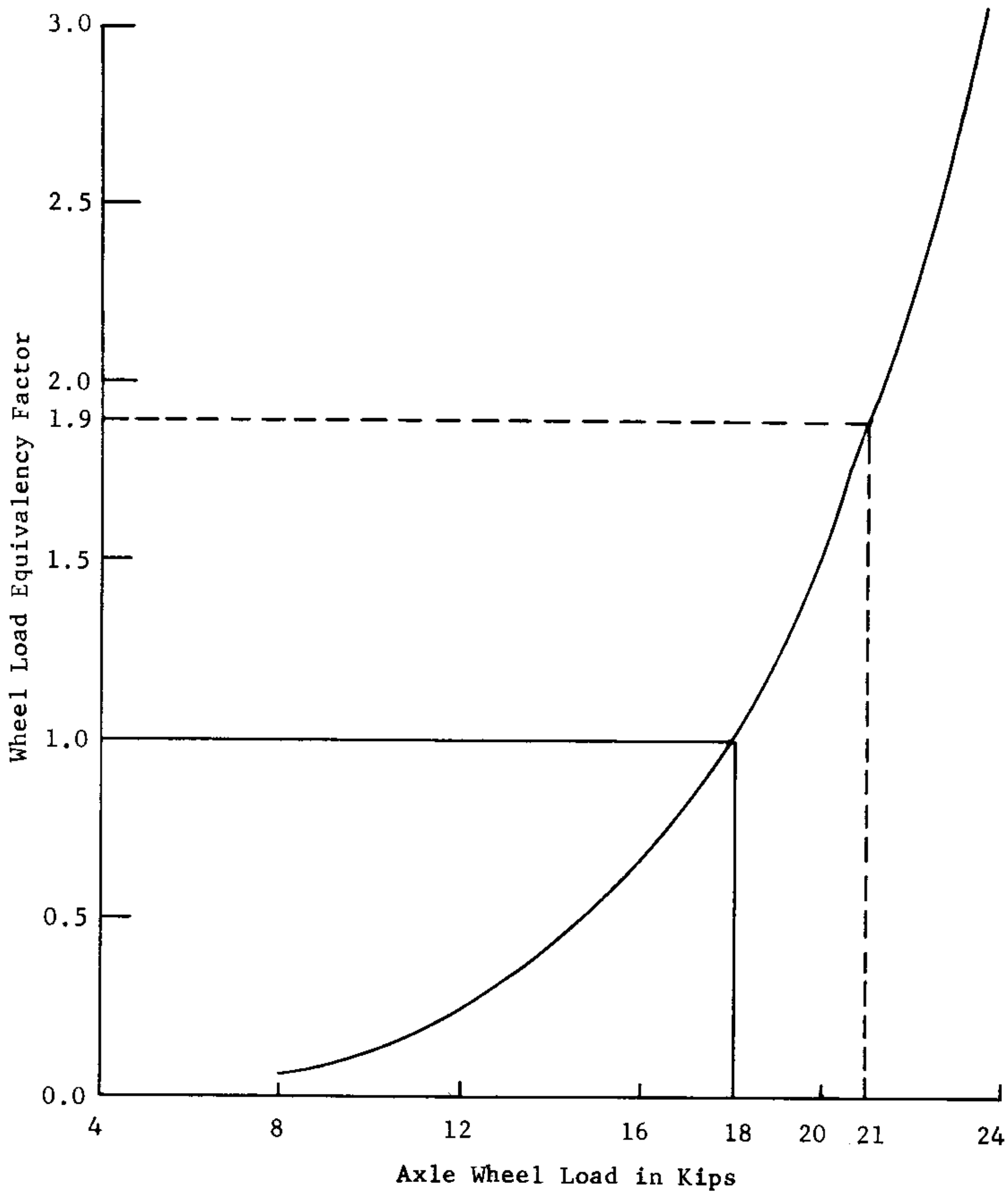


FIGURE A-2
RELATION BETWEEN WHEEL LOAD EQUIVALENCY FACTOR AND AXLE WHEEL LOAD.

Source: AASHO, Interim Guide for the Design of Flexible Pavement Structures, Washington, D. C. 1961.

TABLE A-1

CORRECTIONS APPLIED TO BENKELMAN BEAM READINGS (A SAMPLE)

Ring #3 - Section 1 - 0.0 inches ATB[1]

Date	Pavement Temperatures	Deflection at Pavement Temperatures	Correction Adjustment Factor	Deflection at 70°F	Wheel Load Applications	Equivalent Wheel Load Applications
09-11-67	66.20	0.038	1.10	0.04180	138	263
09-18-67	100.16	0.045	0.16	0.02970	72,366	137,495
09-19-67	104.00	0.047	0.64	0.03000	116,826	221,969
09-22-67	71.00	0.060	0.98	0.05880	153,000	290,700
09-25-67	73.70	0.058	0.93	0.05396	199,437	378,930
09-27-67	71.00	0.062	0.98	0.06076	240,891	457,692
10-09-67	68.50	0.056	1.04	0.05824	325,971	619,344
10-13-67	55.00	0.052	1.42	0.07384	396,000	752,685
10-16-67	55.20	0.055	1.42	0.07810	442,000	840,624
10-24-67	47.00	0.050	1.72	0.08600	543,471	1,032,594
10-31-67	57.33	0.056	1.36	0.07616	595,800	1,132,020
11-06-67	48.00	0.061	1.68	0.10248	640,000	1,216,300

Section 1 had no base, while the other sections had bases of varying thicknesses.

[1]Asphalt Treated Base--special aggregate hot mix.

NOTE: For all the 24 sections these types of calculations were made.

APPENDIX B

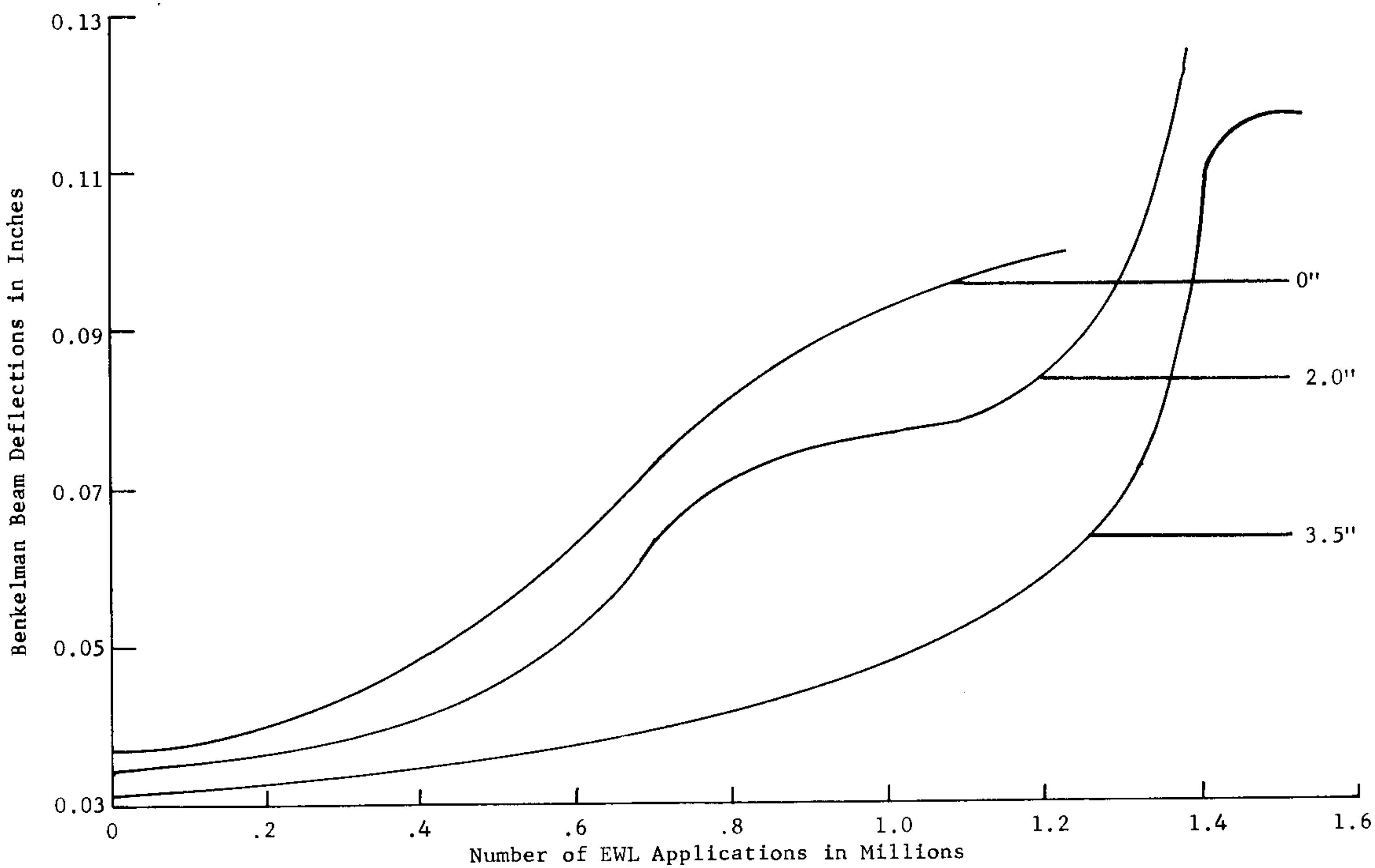


FIGURE B-1
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR SPECIAL AGGREGATE ASPHALT TREATED BASE (RING #3)

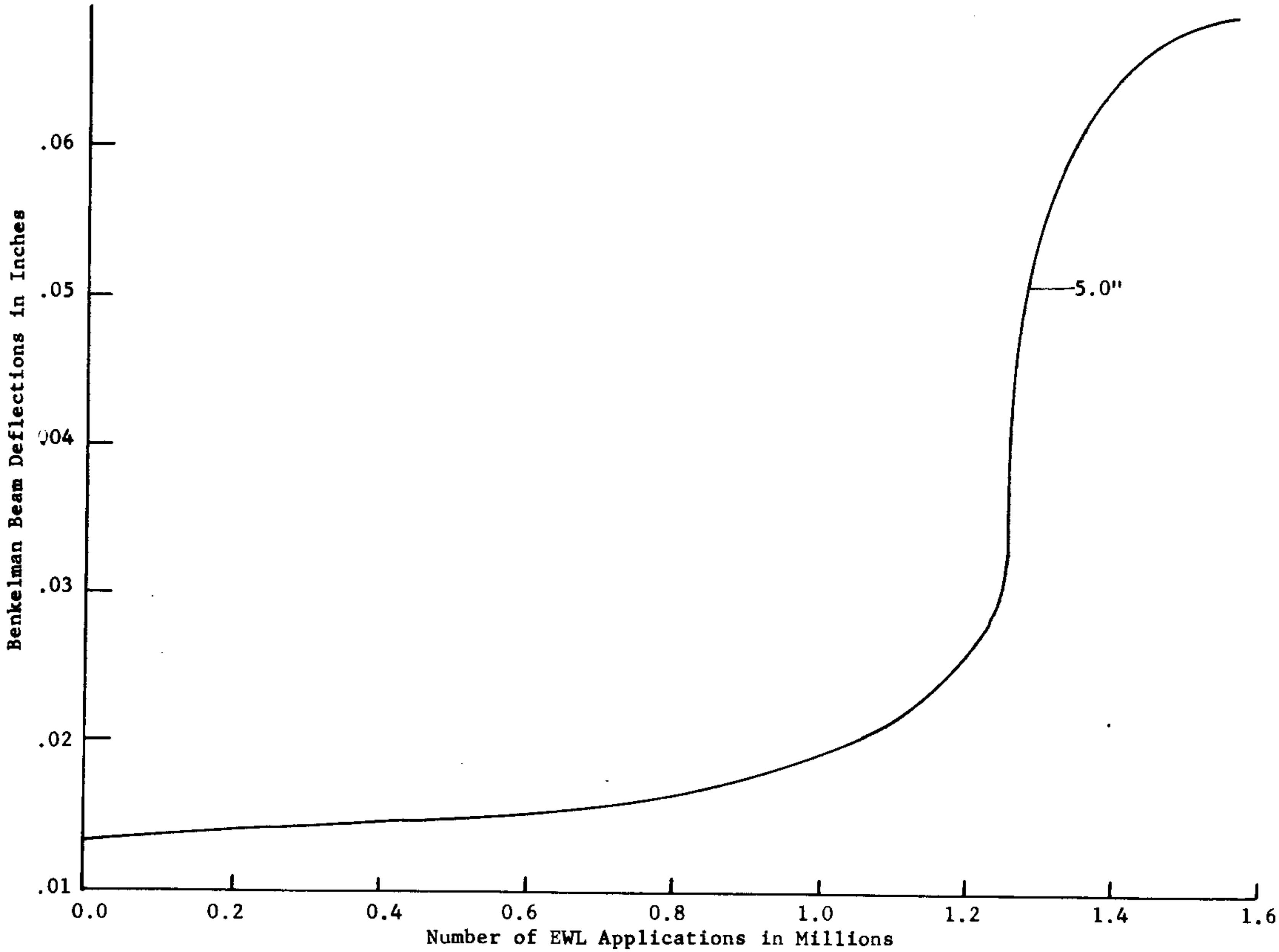


FIGURE B-2
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR SPECIAL AGGREGATE ASPHALT TREATED BASE (RING #3)

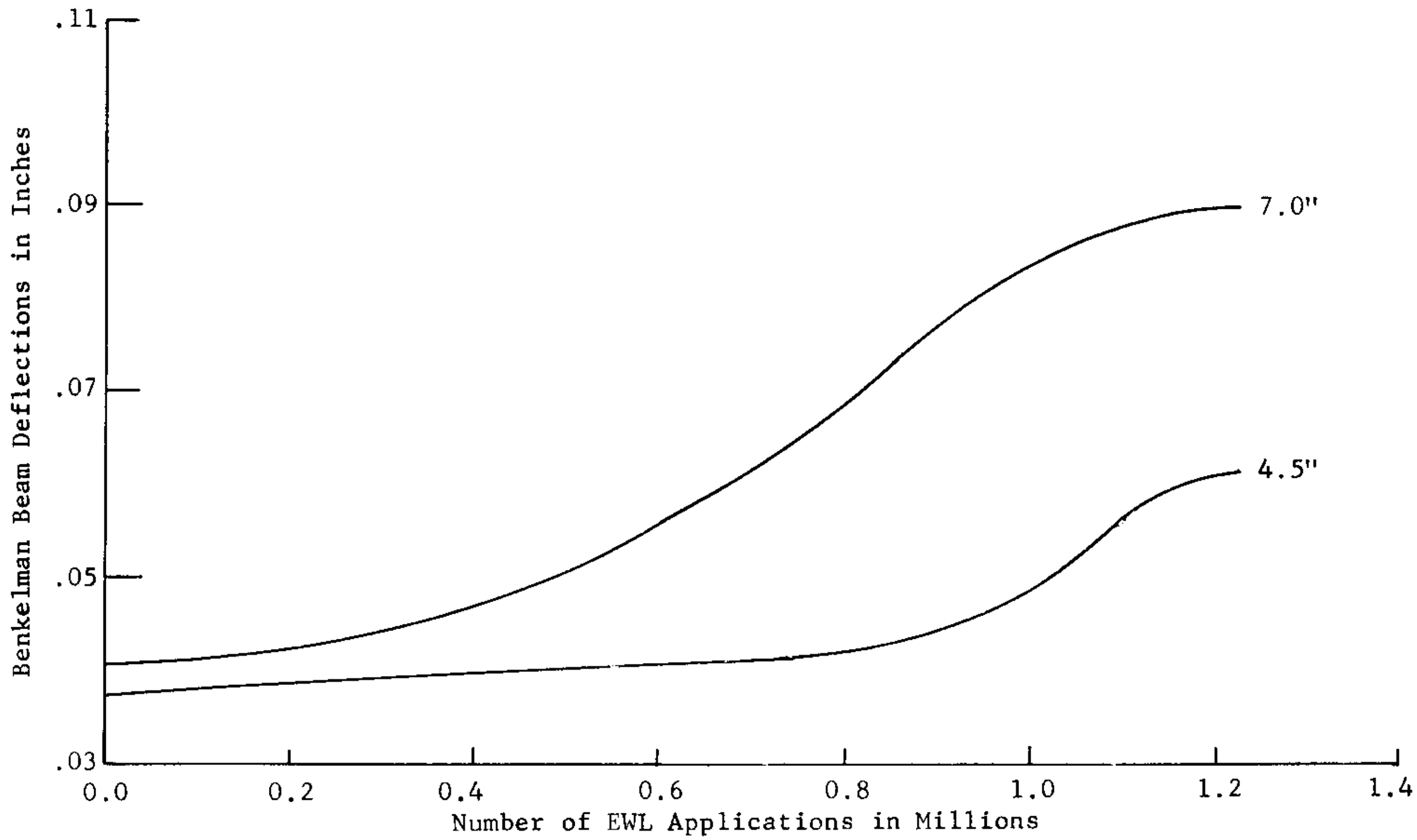


FIGURE B-3
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR UNTREATED CRUSHED ROCK BASE (RING #3)

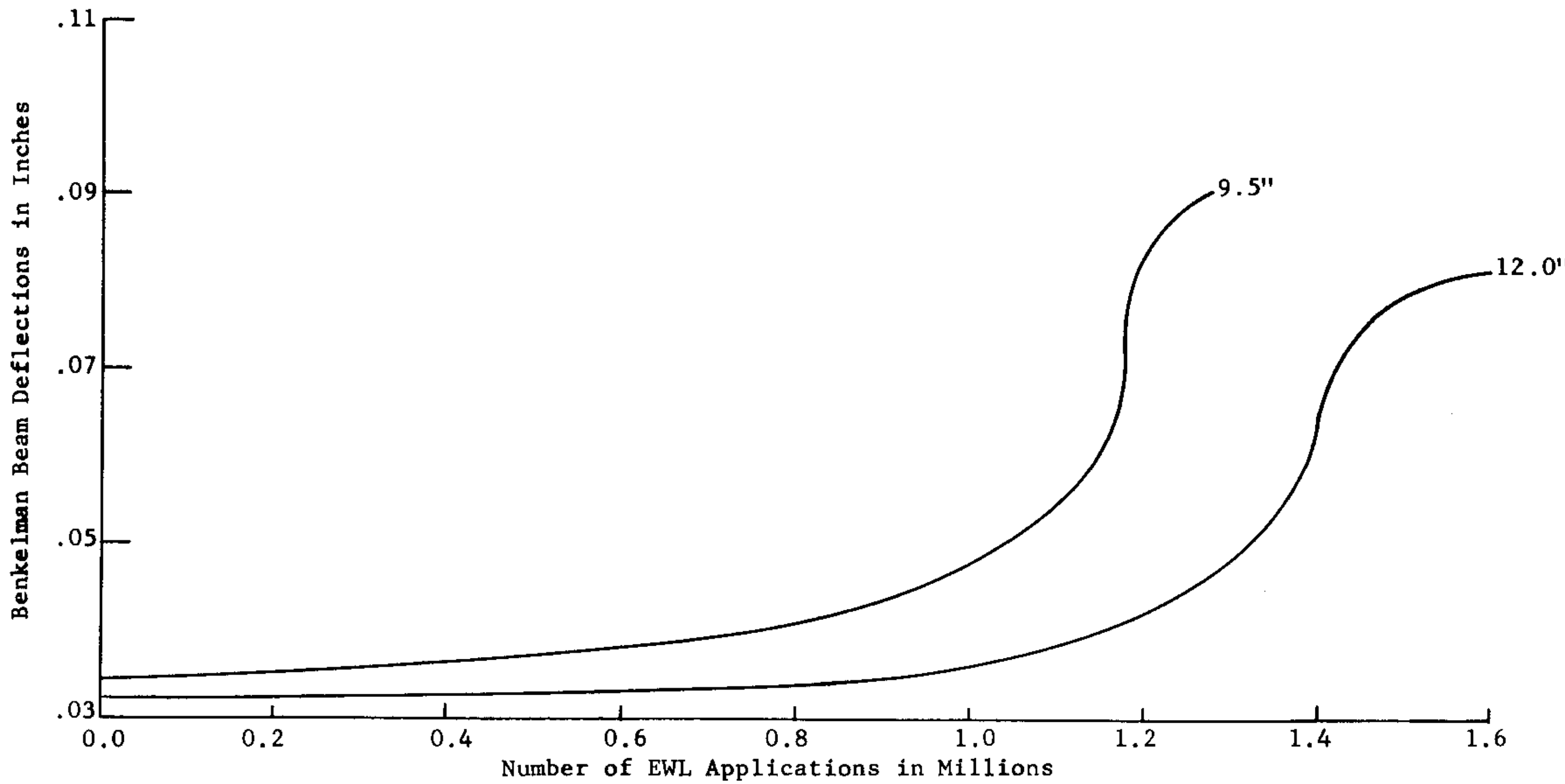


FIGURE B-4
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR UNTREATED CRUSHED ROCK BASE (RING #3)

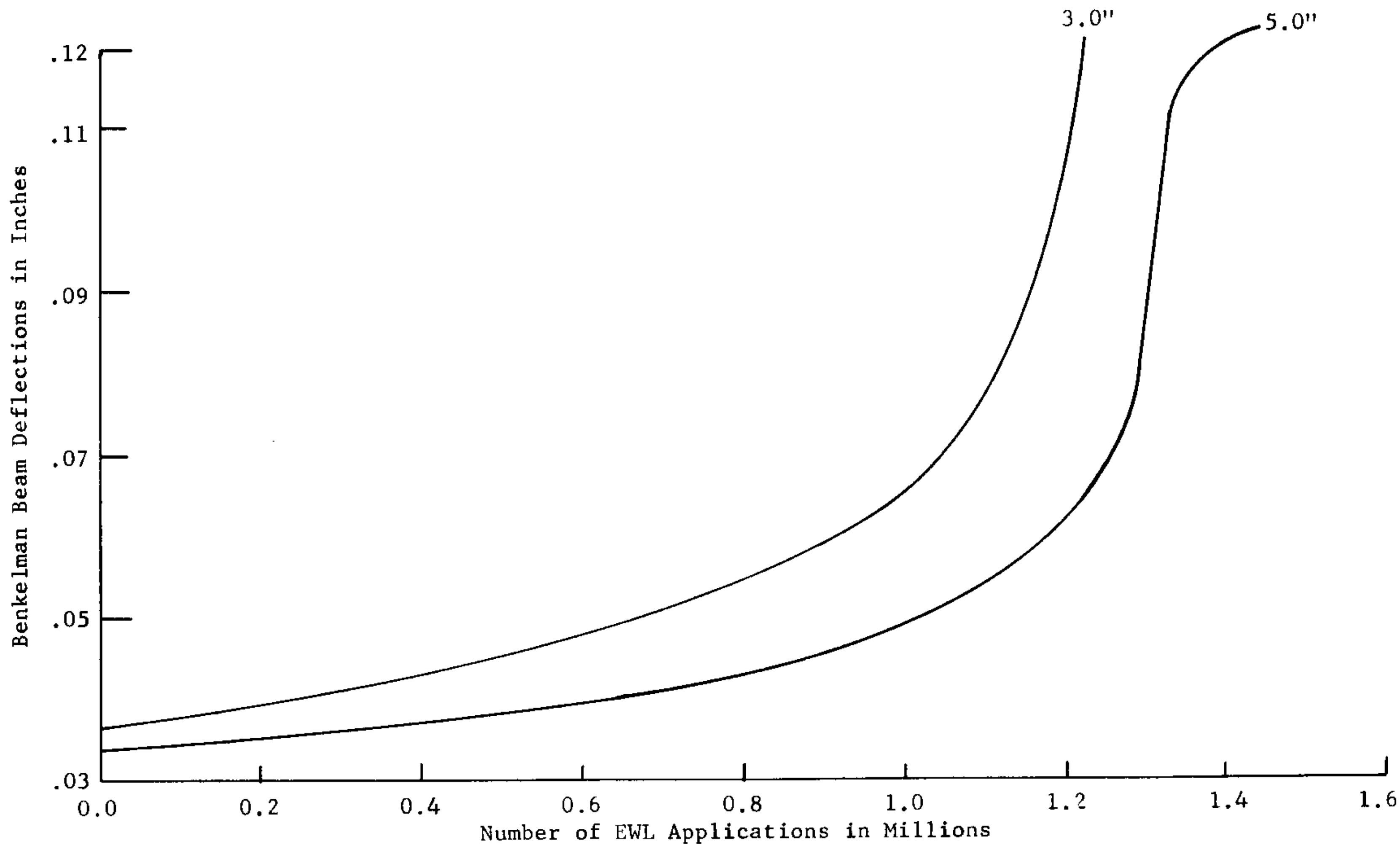


FIGURE B-5
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR EMULSION ASPHALT TREATED BASE (RING #3)

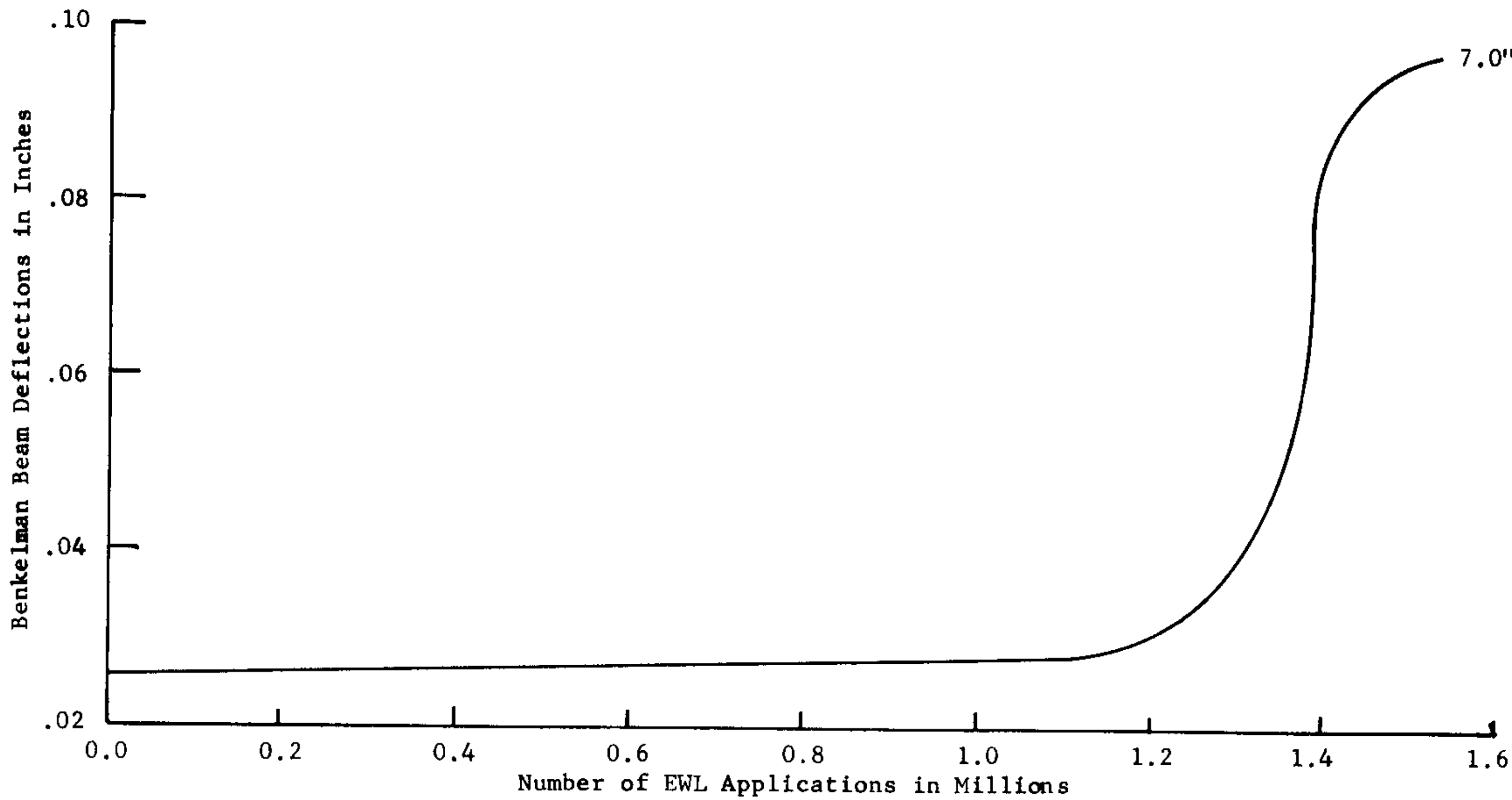


FIGURE B-6
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR EMULSION ASPHALT TREATED BASE (RING #3)

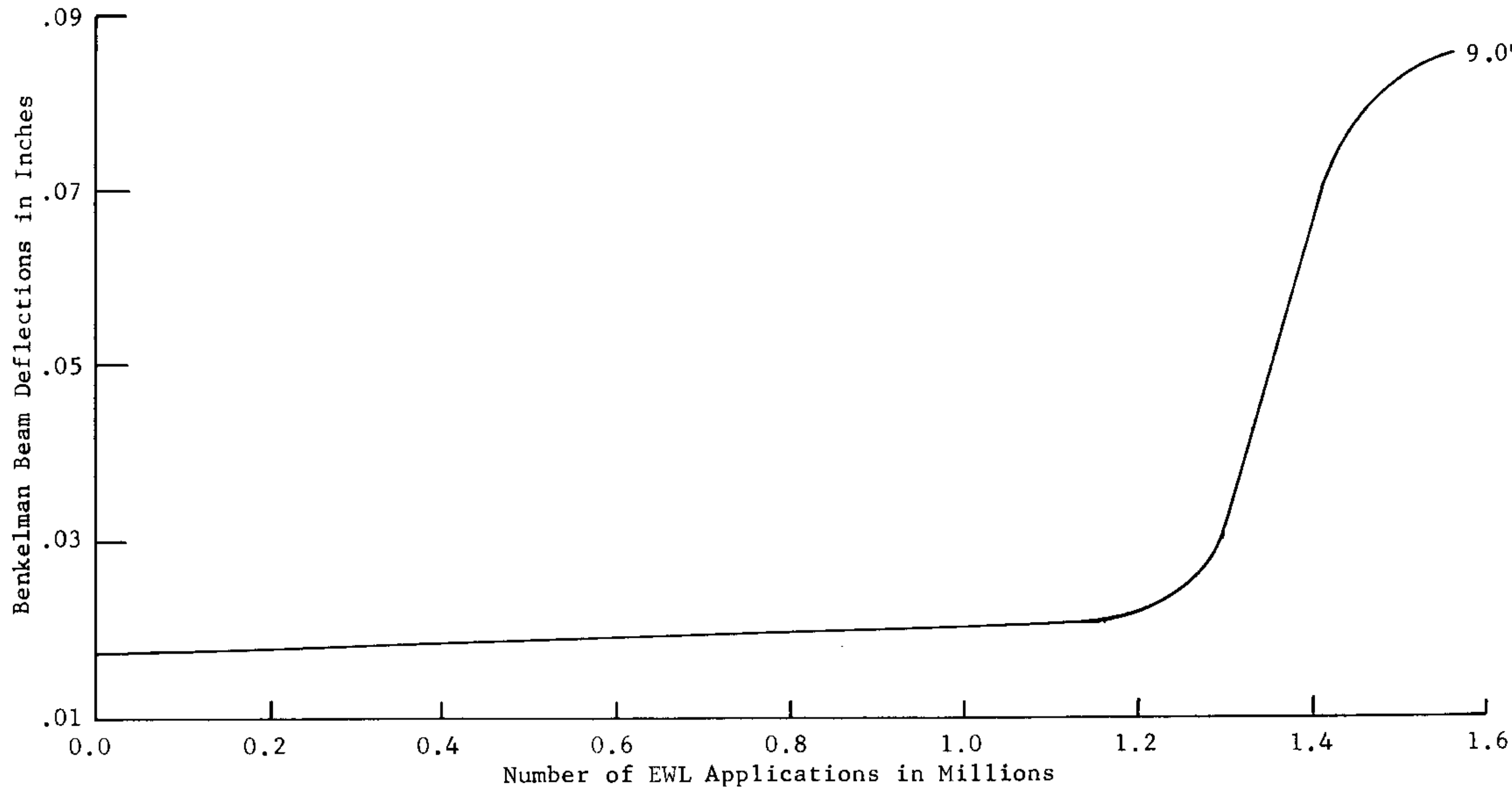


FIGURE B-7
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR EMULSION ASPHALT TREATED BASE (RING #3)

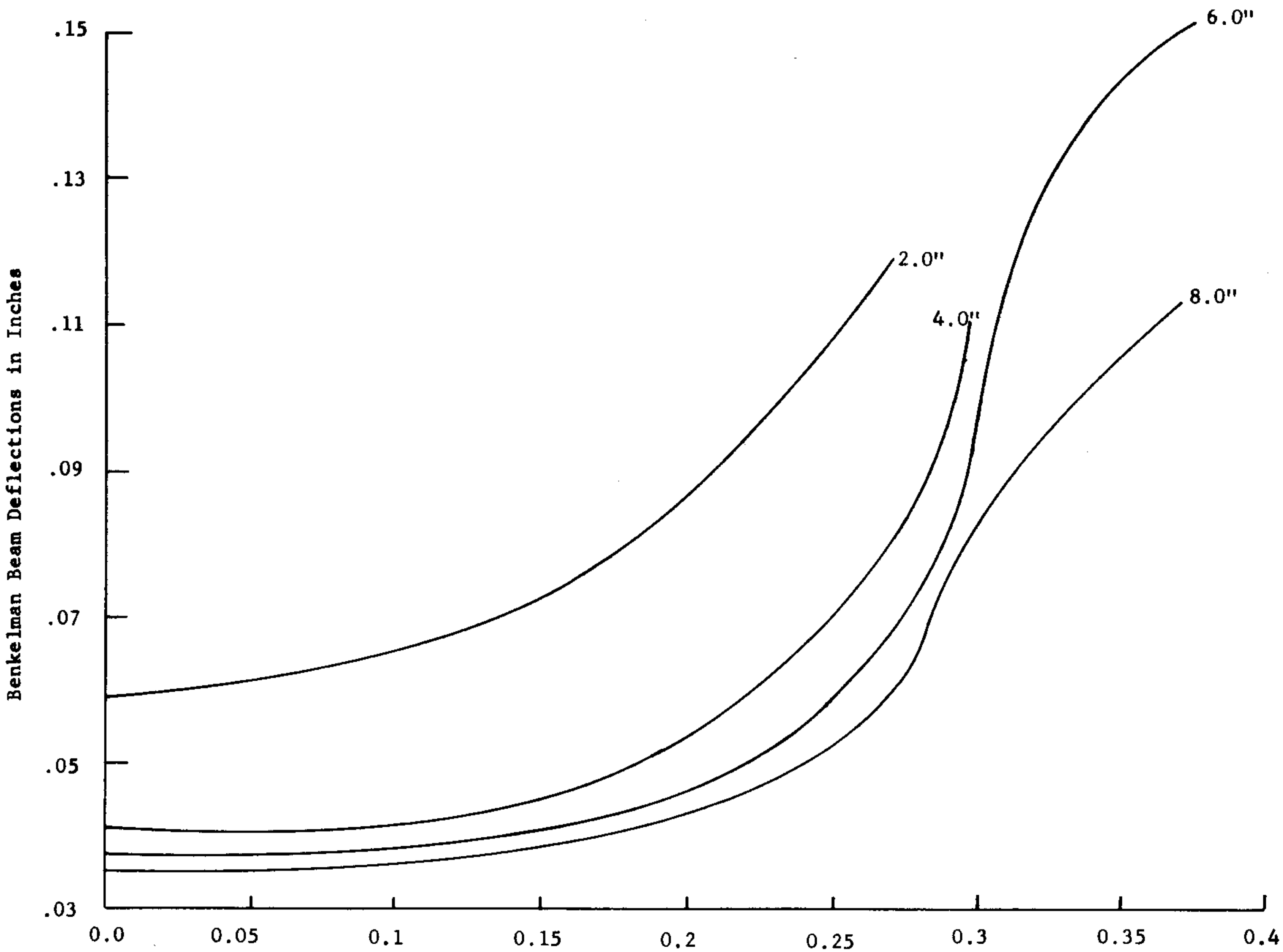


FIGURE B-8
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS FOR SAND ASPHALT BASE (RING #4).

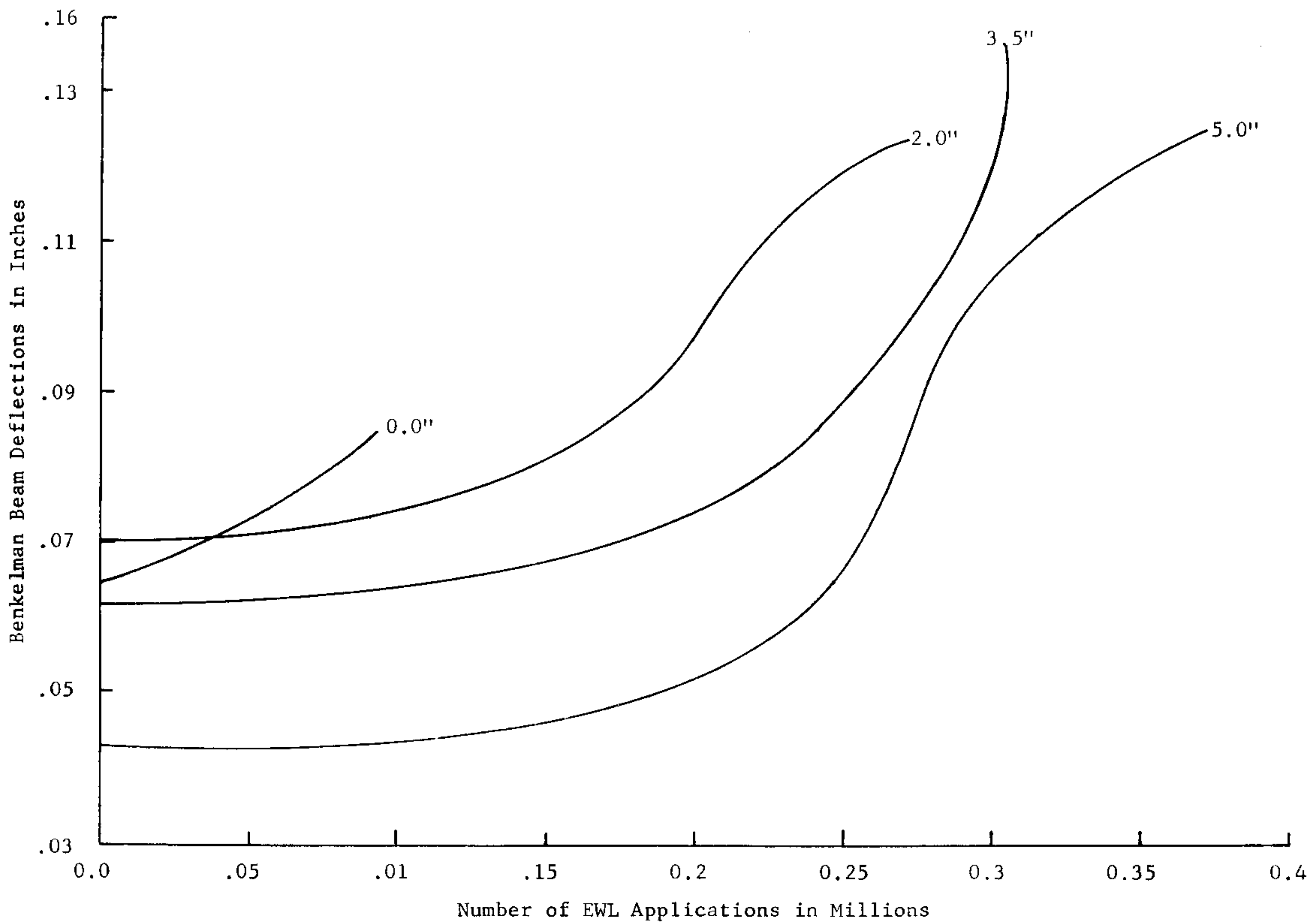


FIGURE B-9
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS
FOR CLASS "F" ASPHALT TREATED BASE (RING #4)

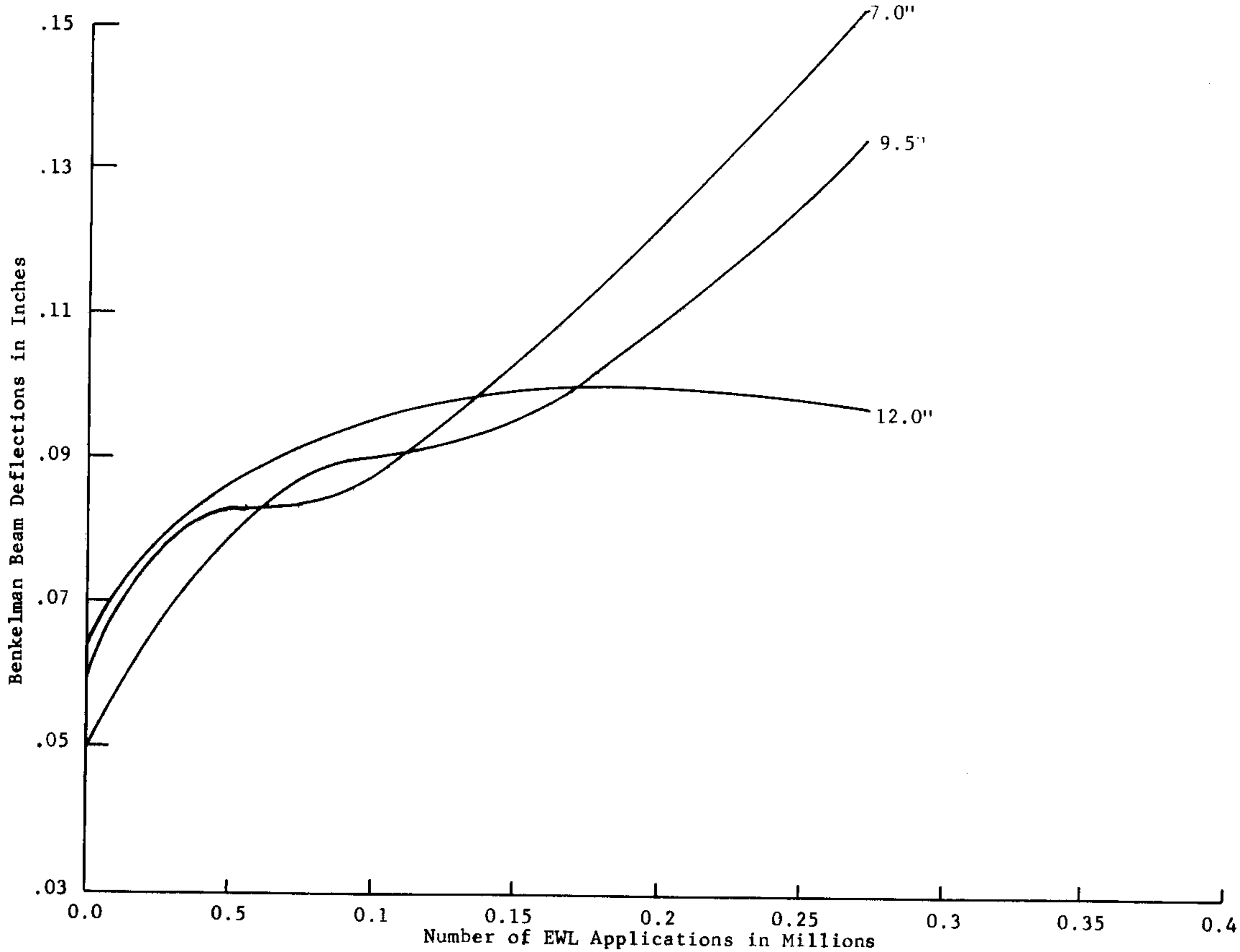


FIGURE B-10
RELATION BETWEEN BENKELMAN BEAM DEFLECTION AND EWL APPLICATIONS FOR UNTREATED CRUSHED ROCK BASE (RING #4)

APPENDIX C

THEORY OF INTERPOLATION

The interpolation discussed in the following lines is based on three dimensional geometry. Here the X axis represents the number of EWL applications, the Y and Z axis represents the thickness of the base and Benkelman Beam deflection, respectively.

Refer to Figure C-1.

Assume the coordinates of the points to be:

$x_2 = W_f$ = EWL applications at failure

$y_2 = T_f$ = Thickness of the base

$z_2 = D_f$ = BB deflection at failure

Assume that a straight line exists which passes through the origin and the point--x_2, y_2, z_2.

For such a condition:

$$d = \sqrt{(x_2 - 0)^2 + (y_2 - 0)^2 + (z_2 - 0)^2} = \sqrt{x_2^2 + y_2^2 + z_2^2}$$

When x_2 is very large as compared to y_2 and z_2, d reduces to:

$$d = \sqrt{x_2^2} = x_2$$

The direction cosines are as:

$$\text{Cos A} = \frac{x_2 - x_1}{d}, \quad \text{Cos B} = \frac{y_2 - y_1}{d}, \quad \text{Cos C} = \frac{z_2 - z_1}{d}$$

When x = y = z = 0 (passing through origin), then direction cosines are:

$$\text{Cos A} = \frac{x_2}{d}, \quad \text{Cos B} = \frac{y_2}{d}, \quad \text{Cos C} = \frac{z_2}{d},$$

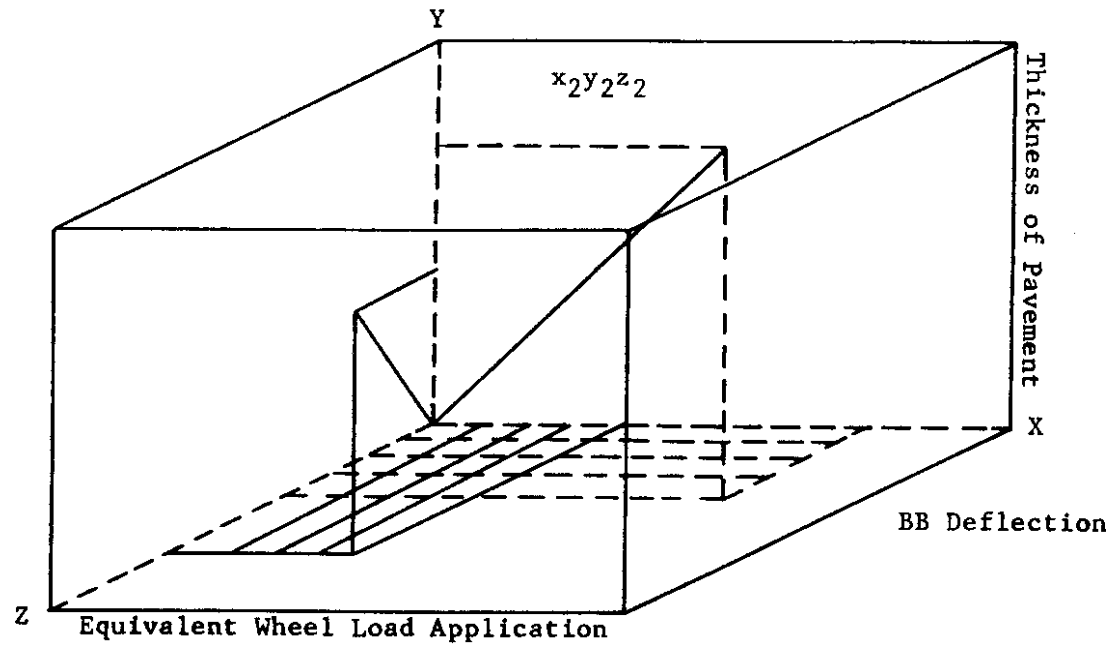


FIGURE C-1
DIAGRAM SHOWING THE BASIS OF THREE DIMENSIONAL APPROACH FOR INTERPOLATION.

The equation of any line passing through origin and (x_2, y_2, z_2) is given as:

$$\frac{x - x_2}{\text{Cos } A} = \frac{y - y_2}{\text{Cos } B} = \frac{z - z_2}{\text{Cos } C}$$

When $z = z_2$ the equation reduces to:

$$\frac{x - x_2}{\text{Cos } A} = \frac{y - y_2}{\text{Cos } B} \quad \text{or} \quad y - y_2 = \left[\frac{x - x_2}{\text{Cos } A}\right] \text{Cos } B$$

$$\text{or } y = y_2 + \left[\frac{x - x_2}{\text{Cos } A}\right] \text{Cos } B$$

Changing the nomenclature and replacing it with:

$x = W_e$ = Expected EWL applications

$y = T_e$ = Expected thickness of pavement

$z = D_e$ = Expected deflection at W_e and T_e

We have:

$$T_e = T_f + (W_e - W_f)\frac{T_f}{W_f} \qquad \text{or}$$

$$T_e = T_f\left[1 + \frac{W_e - W_f}{W_f}\right] \qquad \text{When } D_e = D_f \qquad \text{(A)}$$

Similarly it can be proved that:

$$W_f = W_e\left[1 + \frac{D_f - D_e}{D_e}\right] \qquad \text{When } T_e = T_f \qquad \text{(B)}$$

These two equations are to be used in conjunction with the graphs (Figures 5 - 8). Equation (A) will give the expected safe thickness for expected EWL applications depending upon the data read-off from the graphs. The equation (B) will give the number of EWL applications at which the pavement may fail depending upon the Benkelman Beam deflection, when the thickness of the pavement is known and is constant.

APPENDIX D

TABLE D-1

SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

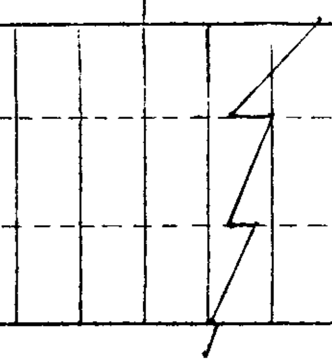
(After R. L. Terrel)

Ring No: 2	**No. of Wheel Load Applications:** 100,000
Section No:	
Base Type: UTB	Date: 11/14/66
Base Thickness:	**Trial No:** 11 of 11

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	7.3 - 20.3	9.3	6.0
- deep	2.3 - 10.7	39.0	33.6
Benkelman Beam	6.5 - 6.5	8.0	33.6
Longitudinal Strain (in. x 10^{-6})			
Surface	---	---	306 C
Top of Base	6.0 - 7.0	60 C	306 T
Top of Subgrade	5.3 - 7.7	40 T	406 T
Vertical Pressure (psi)			
Top of Base - WSU Cell	---	---	----
- Filpip	1.3 - 14.3	57.6**	15.7
Top of Subgrade - WSU Cell	---	---	----
- Filpip	1.3 - 14.3	8.4	10.1

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
		0		
Asphalt Concrete			/E*/ = 1,100,000	/E*/ = 1,100,000
		3.0		
UTB			M_R = 17,000	M_R = 15,500
		5.0		
UTB			M_R = 13,000	M_R = 13,000
		7.0		
UTB			M_R = 11,000	M_R = 10,900
		10.0		
Subgrade			M_R = 9,000	M_R = 9,000



Tension ⟵ Stress ⟶ Compression
(1 division = 10 psi)

*Midway between wheels **Very irregular readings.

TABLE D-2

SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

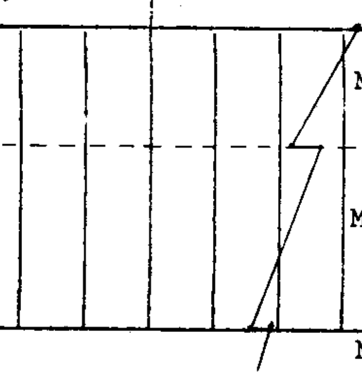
(After R. L. Terrel)

Ring No: 6 Section No: Base Type: ETB Base Thickness: 5.0"	**No. of Wheel Load Applications:** 100,000 Date: 11/14/66 Trial No: 14 of 14

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	5.3 - 7.7	17.6	4.0
- deep	4.9 - 8.1	35.0	33.7
Benkelman Beam	6.5 - 6.5	14.0	33.7
Longitudinal Strain (in. x 10^{-6})			
Surface	10.1 - 2.9	15 C	309 C
Top of Base	-----	----	315 T
Top of Subgrade	-----	----	436 T
Vertical Pressure (psi)			
Top of Base - WSU Cell	12.5 - 0.5	2.0	17.4
- Filpip	5.1 - 7.9	6.0	17.4
Top of Subgrade - WSU Cell	-----	---	11.5
- Filpip	5.1 - 7.9	14.0	11.5

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
Asphalt Concrete		0	/E*/ = 1,100,000	/E*/ = 1,100,000
ETB		3.0	M_R = 17,300	M_R = 17,500
ETB		5.0	M_R = 13,600	M_R = 14,700
Subgrade		8.0	M_R = 9,000	M_R = 9,000



Tension ⟵ Stress ⟶ Compression

(1 division = 10 psi)

TABLE D-3

SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

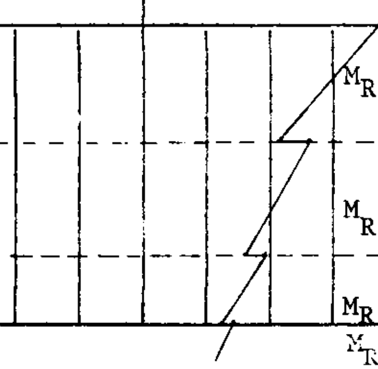
(After R. L. Terrel)

Ring No:	No. of Wheel Load Applications: 100,000
Section No: 8	Date: 11/14/66
Base Type: ETB	
Base Thickness: 9.0"	Trial No: 11 of 11

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	14.3 - 1.3	5.5	6.5
- deep	15.5 - 2.5	13.0	30.0
Benkelman Beam	6.5 - 6.5	9.0	32.8
Longitudinal Strain (in. x 10^{-6})			
Surface	3.5 - 9.5	160 C	304 C
Top of Base	22.1 - 9.1	100 T	167 T
Top of Subgrade	22.7 - 9.7	20 T	264 T
Vertical Pressure (psi)			
Top of Base - WSU Cell	-----	---	----
- Filpip	16.1 - 3.1	0.2	13.4
Top of Subgrade - WSU Cell	-----	---	----
- Filpip	16.1 - 3.1	2.8	6.8

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
		0		
Asphalt Concrete			/E*/ = 1,100,000	/E*/ = 1,100,000
		3.0		
ETB			M_R = 17,500	M_R = 18,500
		6.0		
ETB			M_R = 13,500	M_R = 15,000
		9.0		
ETB			M_R = 11,000	M_R = 12,700
Subgrade		12.0	M_R = 9,000	M_R = 9,000



Tension ⟵ Stress ⟶ Compression

(1 division = 10 psi)

TABLE D-4

SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

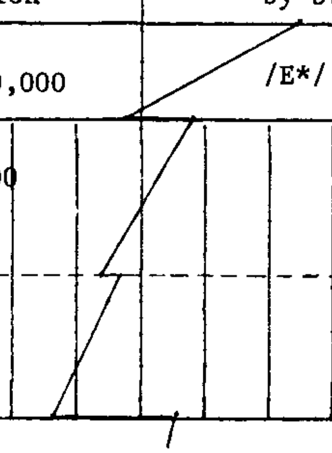
(After R. L. Terrel)

Ring No: Section No: 10 Base Type: ATB Base Thickness: 3.5"	No. of Wheel Load Applications: 100,000 Date: 11/14/66 Trial No: 6 of 6

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	10.1 - 2.9	1.0	0.24
- deep	7.7 - 5.3	21.0	20.3
Benkelman Beam	6.5 - 6.5	8.0	20.3
Longitudinal Strain (in. x 10^{-6})			
Surface	18.5 - 5.5	100 C	111 C
Top of Base	17.3 - 4.3	80 T	8 T
Top of Subgrade	16.8 - 3.8	80 T	129 T
Vertical Pressure (psi)			
Top of Base - WSU Cell	-----	---	---
- Filpip	13.1 - 0.1	---	47.8
Top of Subgrade - WSU Cell	-----	---	---
- Filpip	13.1 - 0.1	2.8	5.3

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
		0		
Asphalt Concrete			/E*/ = 1,100,000	/E*/ = 1,100,000
		3.0		
ATB			/E*/=1,200,000	/E*/=1,200,000
		4.5		
ATB			/E*/=800,000	/E*/=800,000
		6.5		
Subgrade			M_R = 9,000	M_R = 9,000



Tension ⟵ Stress ⟶ Compression
(1 division = 50 psi)

TABLE D-5

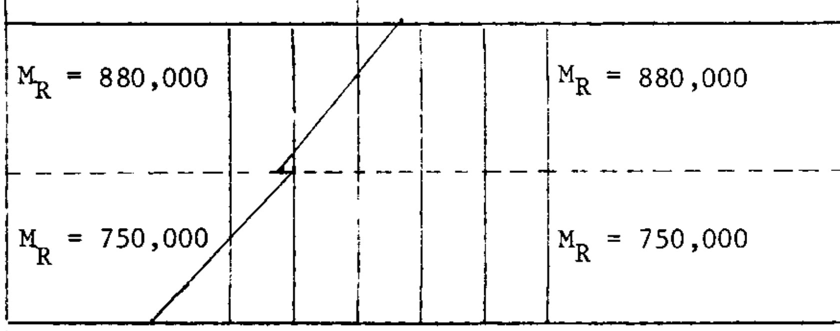
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 3 Section No: 3 Base Type: ATB Base Thickness: 3.5"	No. of Wheel Load Applications: 339,000 Date: 10-10-67 Trial No: 4

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	6.0 - 14.0	0.37	0.024 (55°)
- deep	1.7 - 14.7	26.78	22.51 (53.5°)
Benkelman Beam	6.5 - 6.5	30.00	23.42 (57°)
Longitudinal Strain (in. x 10^{-6})			
Surface	3.8 - 16.8	65 C	87.2 C
Top of Base	3.6 - 16.6	10 T	0.04 C
Top of Subgrade	2.6 - 15.6	60 T	98.7 T
Vertical Pressure (psi)			
Top of Base	6.5 - 6.8	---	8.55
Top of Subgrade	2.9 - 10.1	5.0	5.73

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	55	0		
Asphalt Concrete			/E*/ = 800,000	/E*/ = 800,000
	53.5	3.0		
ATB			M_R = 880,000	M_R = 880,000
		4.5		
ATB			M_R = 750,000	M_R = 750,000
Subgrade	57	6.5	M_R = 8,000	M_R = 8,200



Bulk
Tension ⟵ Stress ⟶ Compression
(1 division = 100 psi)

TABLE D-6
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 3
Section No: 6
Base Type: UTB
Base Thickness: 7.0"

No. of Wheel Load Applications: 339,000
Date: 10-10-67
Trial No: 5

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	8.1 - 21.1	5.6	3.29
- deep	5.5 - 18.5	26.78	34.83
Benkelman Beam	6.5 - 6.5	44.8	43.74
Longitudinal Strain (in. x 10^{-6})			
Surface	5.3 - 18.3	130 C	166.5 C
Top of Base	6.5 - 6.5	---	47.2 T
Top of Subgrade	6.5 - 6.5	---	388.2 T
Vertical Pressure (psi)			
Top of Base	6.5 - 6.5	---	17.3
Top of Subgrade	6.0 - 19.0	6.5	6.9

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	57	0		
Asphalt Concrete			/E*/ = 700,000	/E*/ = 700,000
	55.5	3.0		
UTB			M_R = 15,000	M_R = 18,500
		5.0		
UTB			12,000	15,500
		7.0		
UTB			10,000	13,000
	64.5	10.0		
Subgrade			M_R = 7,000	M_R = 6,900

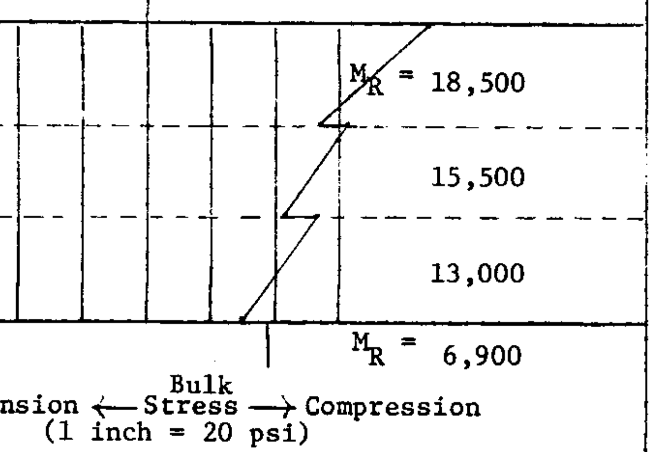


TABLE D-7

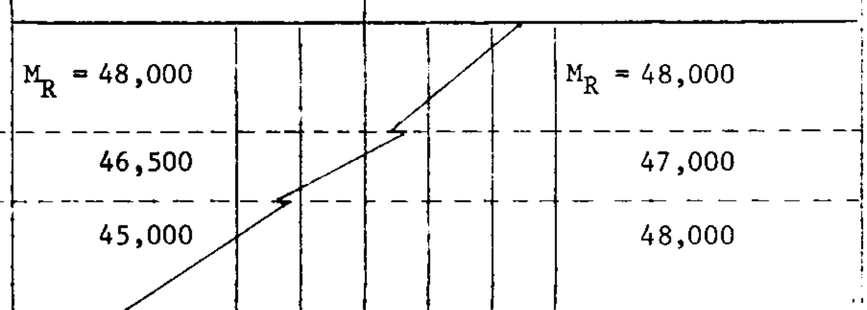
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 3	**No. of Wheel Load Applications:** 339,000
Section No: 10	**Date:** 10-10-67
Base Type: ETB	**Trial No:** 4
Base Thickness: 5.0	

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	2.5 - 10.5	2.33	2.76
- deep	1.7 - 11.3	22.85	33.96
Benkelman Beam	6.5 - 6.5	35.00	34.12
Longitudinal Strain (in. x 10^{-6})			
Surface	0.5 - 12.5	125 C	216.8 C
Top of Base	1.5 - 11.5	100 T	172.8 T
Top of Subgrade	6.5 - 6.5	---	310.4 T
Vertical Pressure (psi)			
Top of Base	0.5 - 6.5	---	18.94
Top of Subgrade	5.3 - 8.7	9.0	7.85

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	62.5	0		
Asphalt Concrete			/E*/ = 640,000	/E*/ = 640,000
	55.5	3.0		
ETB			M_R = 48,000	M_R = 48,000
		4.5		
ETB			46,500	47,000
		6.0		
ETB			45,000	48,000
Subgrade	61	8.0	M_R = 8,000	M_R = 8,000



Tension ⟵ Stress ⟶ Compression

(1 division = 10 psi)

Comment: There is tension on bottom layers of ETB. More trial run necessary

TABLE D-8

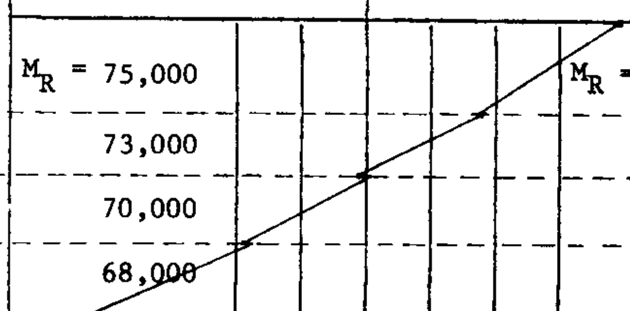
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 3 Section No: 12 Base Type: ETB Base Thickness: 9.0	No. of Wheel Load Applications: 339,000 Date: 10-10-67 Trial No: 5

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	4.8 - 17.8	1.12	3.93
- deep	-----	----	
Benkelman Beam	6.5 - 6.5	17.4	24.52
Longitudinal Strain (in. x 10^{-6})			
Surface	7.1 - 20.1	50 C	33.5 C
Top of Base	4.3 - 17.3	20 C	34.9 T
Top of Subgrade	4.0 - 17.0	125 T	144.0 T
Vertical Pressure (psi)			
Top of Base	6.5 - 6.5	---	21.82
Top of Subgrade	4.6 - 17.6	3.2	4.50

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	57	0		
Asphalt Concrete			/E*/ = 800,000	/E*/ = 800,000
	54.5	3.0		
ETB			M_R = 75,000	M_R = 65,000
		5.0		
ETB			73,000	46,000
		7.0		
ETB			70,000	44,000
		9.0		
ETB			68,000	62,000
Subgrade	62	12.0	M_R = 8,000	M_R = 8,000



Tension ⟵ Bulk Stress ⟶ Compression
(1 division = 10 psi)

Comments: Deflection is too high--will have to increase the subgrade modulus and have another trial run.

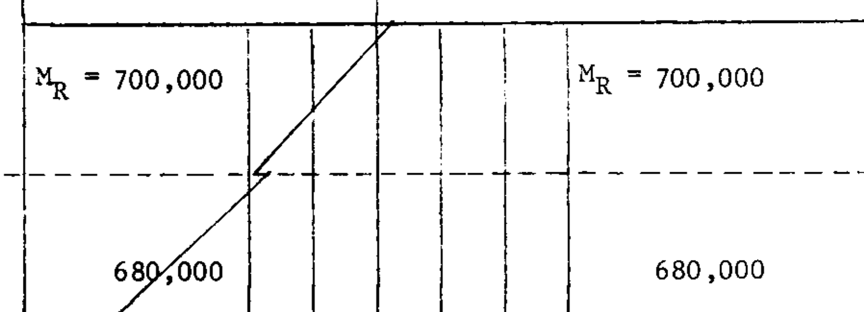
TABLE D-9
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 4	No. of Wheel Load Applications: 37,000
Section No: 2	Date: 11/15/68 (2:30 to 3:30 PM)
Base Type: SAB	
Base Thickness: 4.0"	Trial No: 9

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	3.0 - 10.0	Small	0.34
- deep	3.5 - 16.5	12.1	20.82
Benkelman Beam	6.5 - 6.5	27.0	22.16
Longitudinal Strain (in. x 10^{-6})			
Surface	13.7 - 0.7	120 T	86.8 C
Top of Base	6.5 - 6.5	---	20.6 C
Top of Subgrade	2.0 - 8.5	140 T	137.4 T
Vertical Pressure (psi)			
Top of Base	6.5 - 6.5	---	10.80
Top of Subgrade	6.5 - 6.5	---	4.57

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	33	0		
Asphalt Concrete			/E*/ = 1,200,000	/E*/ = 1,200,000
	35	3.0		
SAB			M_R = 700,000	M_R = 700,000
		5.0		
SAB			680,000	680,000
Subgrade	39	7.0	M_R = 7,500	M_R = 7,800



Bulk
Tension ⟵ Stress ⟶ Compression
(1 inch = 200 psi)

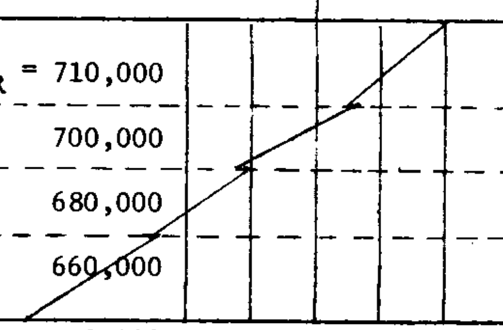
TABLE D-10
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 4	No. of Wheel Load Applications: 37,000
Section No: 4	Date: 11/15-68
Base Type: SAB	Trial No: 3
Base Thickness: 8.0	

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	2.5 - 15.5	---	1.47
- deep	3.0 - 16.0	5.7	14.02
Benkelman Beam	C_L	15.8	13.93
Longitudinal Strain (in. x 10^{-6})			
Surface	19.0 - 6.0	--	15.9 C
Top of Base	19.0 - 6.0	20 C	17.4 C
Top of Subgrade	19.0 - 6.0	70 T	34.5 T
Vertical Pressure (psi)			
Top of Base	C_L		17.0
Top of Subgrade	18.0 - 5.0	1.0	1.86

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	32	0		
Asphalt Concrete			/E*/ = 1,500,000	/E*/ = 1,500,000
	35	3.0		
SAB			M_R = 710,000	M_R = 71,000
		5.0		
SAB			700,000	700,000
		7.0		
SAB			680,000	680,000
		9.0		
SAB			660,000	660,000
Subgrade	40	11.0	M_R = 8,000	M_R = 7,700



Bulk
Tension ← Stress → Compression
(1 inch = 100 psi)

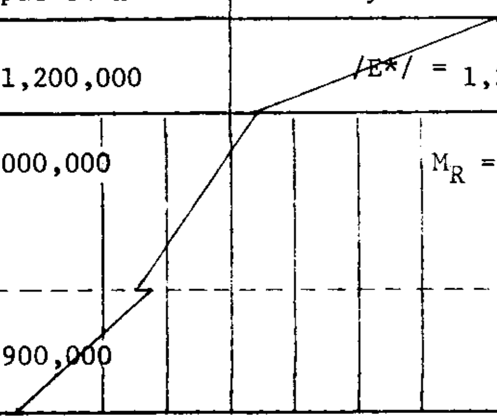
TABLE D-11
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

Ring No: 4 Section No: 7 Base Type: ACB Base Thickness: 3.5"	No. of Wheel Load Applications: 37,000 Date: 11/15/ Trial No: 4

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	3.0 - 10.0	0.74	0.22
- deep	5.0 - 7.0	----	25.03
Benkelman Beam	6.5 - 6.5	30.5	24.84
Longitudinal Strain (in. x 10^{-6})			
Surface	1.0 - 12.0	80 C	94.9 C
Top of Base	0.5 - 12.5	10 C	11.6 T
Top of Subgrade	6.5 - 6.5	--	90.6 T
Vertical Pressure (psi)			
Top of Base	6.5 - 6.5	--	8.57
Top of Subgrade	1.6 - 11.4	6.0	4.27

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	35	0		
Asphalt Concrete			/E*/ = 1,200,000	/E*/ = 1,200,000
	37	3.0		
ACB			M_R = 1,000,000	M_R = 1,000,000
		4.5		
ACB			900,000	900,000
Subgrade	39	6.5	M_R = 6,500	M_R = 6,400



Bulk
Tension ⟵ Stress ⟶ Compression
(1 division = 100 psi)

TABLE D-12
SUMMARY OF PAVEMENT BEHAVIOR PREDICTION

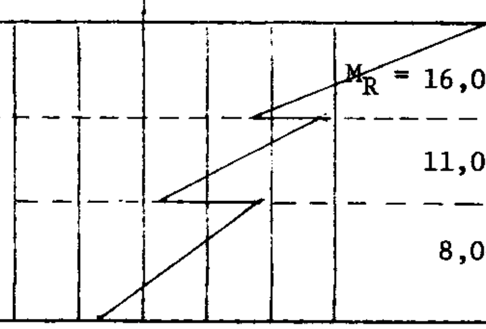
Ring No: 4
Section No: 10
Base Type: UTB
Base Thickness: 7.0"

No. of Wheel Load Applications: 37,000
Date: 11/15/68
Trial No: 6

Item	Wheel Position	Measured	Computed
Deflection (in. x 10^{-3})			
LVDT - shallow	5.0 - 18.0	13.0	4.68
- deep	1.3 - 14.3	35.2	53.83
Benkelman Beam	6.5 - 6.5	58.3	57.18
Longitudinal Strain (in. x 10^{-6})			
Surface	4.0 - 17.0	120 C	131.2 C
Top of Base	4.0 - 17.0	60 T	118.3 T
Top of Subgrade	3.0 - 16.0	140 T	354.1 T
	4.3 - 17.3	210 T	348.2 C
Vertical Pressure (psi)			
Top of Base	6.5 - 66.5	---	12.64
Top of Subgrade	6.0 - 19.0	2.5	5.0

BULK STRESS DISTRIBUTION WITH DEPTH

Material	Temp. °F	Depth in.	Modulus Used for Computation	Modulus Required by Stress Condition
	32	0		
Asphalt Concrete			/E*/ = 1,200,000	/E*/ = 1,200,000
	37	3.0		
UTB			M_R = 16,000	M_R = 16,000
		5.0		
UTB			13,000	11,000
		7.0		
UTB			10,000	8,000
	42	10.0		
Subgrade			M_R = 4,000	M_R = 3,800



Tension ⟵ Stress ⟶ Compression
(1 inch = 10 psi)