COLLEGE OF ENGINEERING
RESEARCH DIVISION
TRANSPORTATION SYSTEMS SECTION

15.6 Research Report No. 73/15-2-75

THE APPLICATION OF WSU TEST TRACK DATA TO FLEXIBLE PAVEMENT DESIGN

by

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November, 1973

Project No. 3808-0918

Contract No.

Sponsor Project No. Y-993 - Supplement #3

Sponsor Contract No. Y-993 - Supplement #3

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Pullman, Washington

PAVEMENT RESEARCH at the WASHINGTON STATE UNIVERSITY TEST TRACK

Volume 6

THE APPLICATION OF WSU TEST TRACK DATA TO FLEXIBLE PAVEMENT DESIGN

Report to the Washington State Department of Highways
on Research Project Y-993 - Supplement #3

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(Transportation Systems Section Publication H-42)

ACKNOWLEDGMENTS

The Transportation Systems staff wishes to thank the Washington State Department of Highways for their financial aid, technical support and patience. The following Highway personnel in Olympia were helpful in many ways: Hollis Goff, Assistant Director for Planning, Research and State Aid; Roger V. LeClerc, Materials Engineer; Ray Dinsmore, Research Coordinator; Mrs. Willa Mylroie, Special Assignments and Research Engineer; Tom R. Marshall, Assistant Materials Engineer; and Carl Toney, Special Assignments Engineer.

Didrik A. Voss, P.E., Senior Engineer and Dr. Ronald L. Terrel, P.E., both of Pavements Systems, International, Seattle, Washington, were very cooperative and helpful as co-authors and consultants on this project. Their advice and help is acknowledged.

Thanks go to our secretary, Cheryl Caraher, for typing and editing. Chuck West, a WSU senior in civil engineering, drafted the figures in this report.

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INTRODUCTION

In 1964, Washington State University constructed a test track for the purpose of testing full scale experimental pavements under actual field conditions encountered in Southeastern Washington. The construction of the track and the first four years of operation were sponsored by the Washington Department of Highways in cooperation with the U.S. Department of Transportation, Federal Highway Administration. The Asphalt Institute contributed both financial and technical aid to the project. Others, such as the Chevron Asphalt Company and the Firestone Tire and Rubber Company, have contributed both materials and technical help during some stages of this project.

During the four years of operation, from 1965 to 1969, four different rings of pavements containing different base materials of varying thicknesses were tested. The results have been published in various reports (1, 2, 3, and 4).[1] Although the results have been partially analyzed, there has been no attempt to try to apply them to flexible pavement design. The overall purpose of this project is to try to apply the various results.

The purpose of this report is to review all of the information collected during the design, construction, and testing of the first four rings and summarize the conclusions and recommendations made. Listed below are specific objectives of this study:

1. Review experiment design and procedures used. Make recommendations on improvements to experiment design, if applicable.

[1] The numbers refer to particular references.

2. Explain performance of test sections in terms of the Hveem Design Method (R-value) and elastic layer theory, and establish criteria if possible.
3. Evaluate pertinent parameters from the test track data for pavement design application.
4. Evaluate field equivalency ratios established for test sections and modify if necessary.

EXPERIMENT DESIGN AND PROCEDURES USED

PHYSIOGRAPHIC DESCRIPTION

The WSU test track is located one mile east of Pullman in Southeastern Washington on a side hill with a south slope. It is adjacent to State Route 270, locally called the Moscow-Pullman Highway.

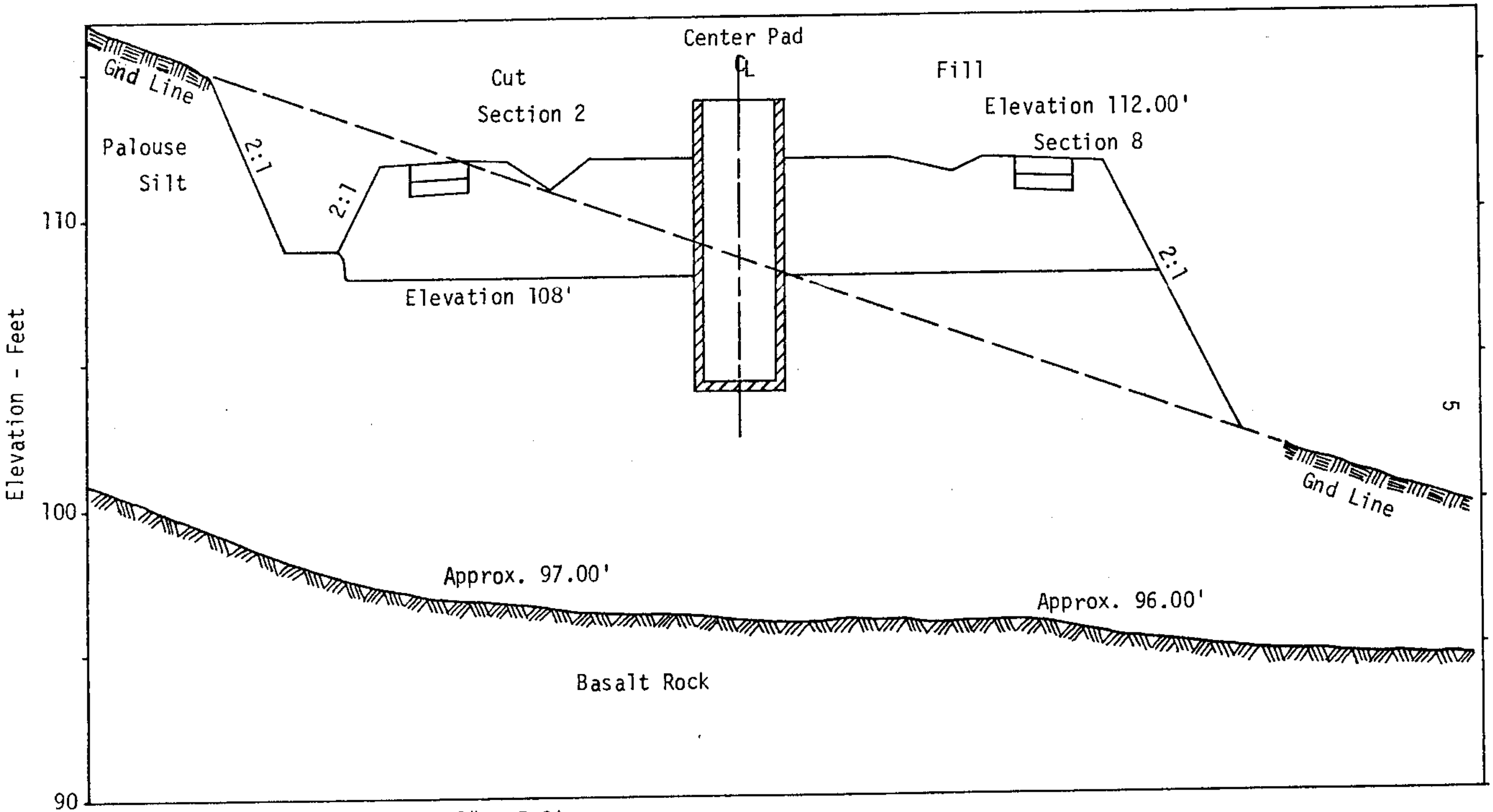
The facility is constructed on Palouse silt, which is a common soil in Southeastern Washington. The engineering properties of this material are described later. The Palouse silt, which lies in a north-south direction beneath the test track, varies in depth from 14-15 feet. Bedrock lies below the silt. The bottom layer of the silt is interfaced with a basalt layer consisting of approximately 2 feet of weathered basalt overlaying columnar and blocky basalt. Water is present at varying depths immediately above the basalt which produces varying moisture contents in the subgrade. The water is fed by means of two natural valleys, one on each side of the test track. Figure 1 is a cross-section of the conditions. The initial subgrade preparation involved approximately equal cut and fill, with a maximum cut depth of 3 feet and a maximum fill depth of 4 feet, to provide a compacted subgrade at elevation 108.0 feet [1]. In later rings the fills were deeper to try to obtain dryer silt subgrade [2, 3, 4].

One solution, to correct this bias, would be to randomly distribute the base types of varying thicknesses. This was possible in Ring 1 where only 6 sections were tested, and 3 were duplicates of each other. Although this alternative of randomly distributing the base types of varying thicknesses was seriously considered, time, costs, and ease of construction dictated that sections of the same base types be grouped together. Random distribution would have required more hand labor and would have prevented the contractor from using standard construction methods [1, 2, 3, 4].

Another solution to the problem of bias would be the following: (1) Construct a concrete platform below the test track as shown in Figure 1 so that the subgrade material is the same depth at all locations. (2) Construct a diversionary ditch and vertical sand drain to insure that water from outside sources does not flow into the test site.

The problem of trying to analyze pavements under varying environments is still in a state of infancy. The present state of the art is to analyze pavements under controlled environment and here more experience is needed. The possibility of constructing a building over the test track that is capable of controlling temperature and moisture in the pavement structure should be seriously considered.

The pavement testing equipment needs some modifications to fully utilize its research potential. The apparatus has the power to run at speeds of up to 45 mph. Unfortunately, the present drive mechanisms have a tendency to leak oil at speeds above 20 mph. The present spring system is inadequate to prevent the apparatus from transferring the shock to the other areas. The drive mechanism and shock system should be modified so that the apparatus could be run at a minimum of 35 mph.



Vertical Scale: 1" = 5.0'
Horizontal Scale: 1" = 20.0'

Figure 1: Cross-section of Test Track showing rock slope under Track.
North-South Direction

The eccentricity mechanism should be improved so that traffic conditions normally found on a road could be duplicated. Under spring conditions, when the subgrade was highly saturated, the movement of the wheels across the pavement was so slow that the pavement immediately under the wheels would fail before other portions of the pavement were loaded. This phenomena will be more adequately explained later during the discussion of rutting. One solution would be to place each set of wheels a different distance from the axes of direction. Another, and perhaps more practical solution, would be to modify the eccentricity drive mechanism by putting in a gear shift or by installing a separate motor to drive the unit. Thus, the eccentricity speed could be varied.

SUMMARY

From the above discussion, the following improvements to the test facility should be made before future experiments are conducted:

1. Construct a concrete platform below the facility so the subgrade is a uniform depth.
2. Construct a diversionary ditch and vertical sand drain to control the intrusion of water into the test site.
3. Construct a building over the test site to control temperature and moisture conditions.
4. Change the drive mechanism and shock absorption system so that the higher speed capability of the apparatus can be fully utilized.
5. Adjust the three sets of wheels so that each is a different distance from the axis of rotation; or increase the speed of the eccentricity mechanism and thus reduce the loading time under the pavements.

DESIGN AND CONSTRUCTION OF TEST RINGS

SUBGRADE

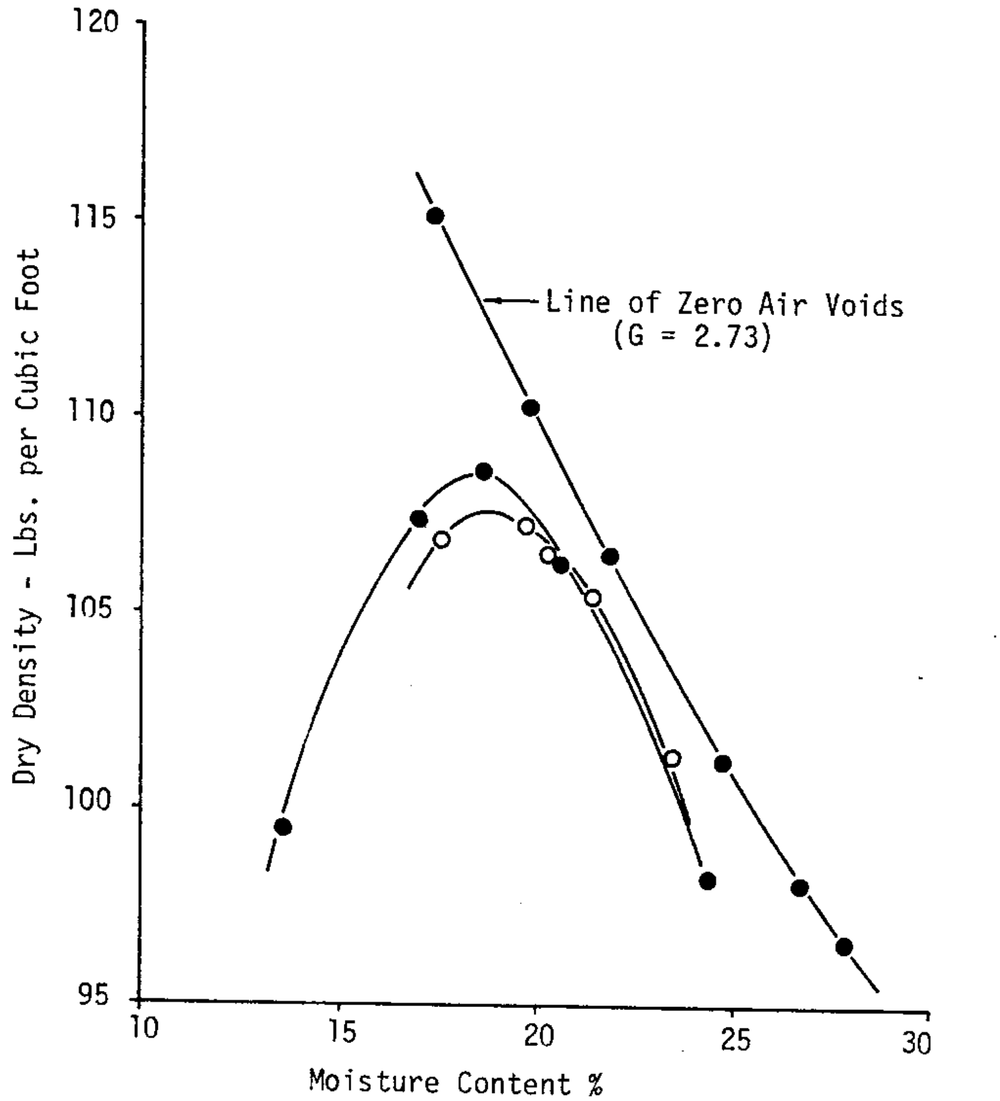
The subgrade soil, a silt clay A-6 (10), is known locally as Palouse silt. This soil covers a wide area in Southeastern Washington. It is a loess, which is a wind-blown deposited material. Figure 2 and Table 1 show the moisture-density relationship. The soil characteristics and classification are listed in Table 2. The R-value tests are listed in Table 3, and the CBR tests are listed in Table 4. Kingham and Kallas [7] determined the dynamic modulus (stiffness), as shown in Figure 3.

Considerable work has been done in trying to determine a relationship between the three methods of determining the strength of materials: R-value, dynamic modulus, and CBR. Figure 4 shows one such relationship. Table 5 lists the relationships found between the three methods for Palouse silt. Although very few samples are used for this comparison, it appears that the relationships shown in Figure 4 [10] are not accurate enough for Palouse silts.

THICKNESS DESIGN

The thicknesses for the test sections of Ring 1 were calculated using the subgrade R-value of 16 and the Washington State Structural Design Chart for Flexible Pavements [11]. The structural sections were constructed to withstand an accumulated traffic loading of 4,000,000 wheel applications. Figure 5 is a schematic of the six sections. Testing began March 1, 1965, was suspended on December 21, 1965, due to bad weather, and resumed February 10, 1966. The test ended on May 20, 1966, after 4,724,100 wheel loads.

At the completion of two million wheel load applications, none of the sections had failed. To accelerate failure, adverse conditions were produced in sections 2, 3, and 4 by saturating the subgrade. The order of failure for



Legend: Subgrade Material

Test Track Site: ●——●

United Paving: ○——○

Note: Standard Proctor
Test used: ASTM - D 698 - 57T A

Figure 2: Density - Moisture Curves For Subgrade Soil

Table 1

OPTIMUM DENSITY AND MOISTURE FOR
PALOUSE SILT SUBGRADE MATERIAL

Source	Max. Optimum Dry Density	Optimum Moisture
Test Track	108.8	18.8
United Paving	107.8	18.8

Table 2

PALOUSE SILT SUBGRADE SOIL CHARACTERISTICS AND CLASSIFICATION

Soil	Specific Gravity S.G.	Liquid Limit L.L.	Plastic Limit P.L.	Plasticity Index P.I.	Highway Res., Board Class.	Airfield Classification
Clay-silt	2.73	34.9	20.2	14.7	A - 6 (10)	CL

Table 3

STABILOMETER TEST RESULTS FOR
PALOUSE SILT SUBGRADE SOIL

Stabilometer Test

cc. Temp. H_2O	A	B	C	D
Cc. H_2O added	75	70	60	82
% H_2O added	7.9	7.3	6.3	8.6
Initial % H_2O	15.4	15.4	15.4	15.4
Molding % H_2O	23.3	22.7	21.7	24.0
Molding Density	96.2	97.5	98.9	93.9
Compactor pressure	100	---	---	---
No. blows	40	---	---	---
Wt. in mold	3170	3181	3173	3174
Wt. in mold (soaked)	3178	3188	3182	3182
Wt. of mold	2176	2190	2164	2163
Net wt. of soil	994	991	1009	1011
Height	254	251	254	263
Exudation pressure	330	400	540	180
Swell pressure	3	17	19	0
Drainage	None	None	None	None
Stabil. "Ph"--500#	22	24	19	26
"--1000#	53	52	39	59
"--2000#	119	118	90	131
Displacement "D"	468	434	418	498
"R" Value	15	16	31	10
Gravel equivalent				
Swell equivalent	1.4	8.2	9.1	0

(Washington State Highway Department,
Materials Laboratory, Olympia, WA)

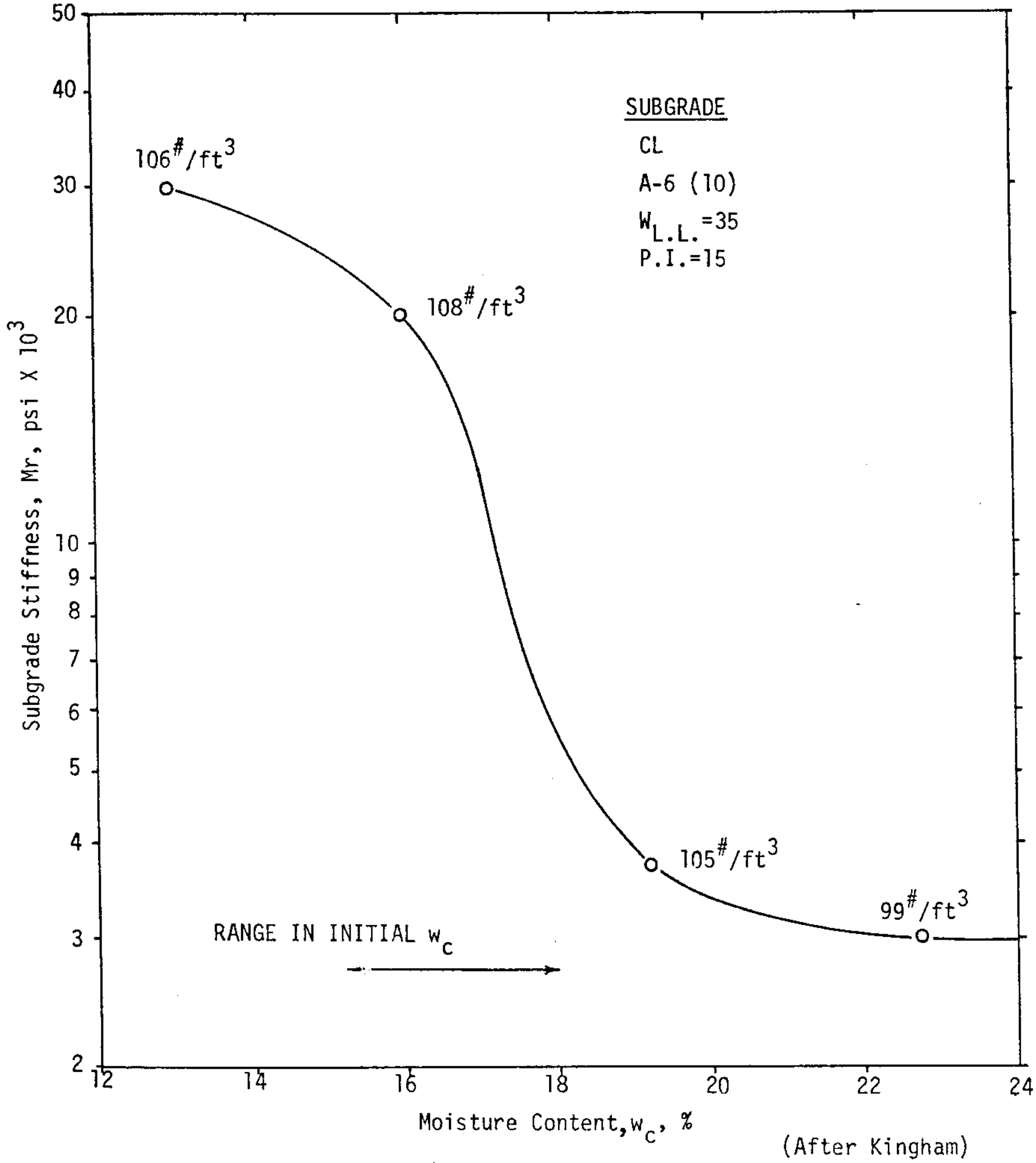


Figure 3: Variation of Laboratory Measured Palouse Silt Subgrade Soil Stiffness

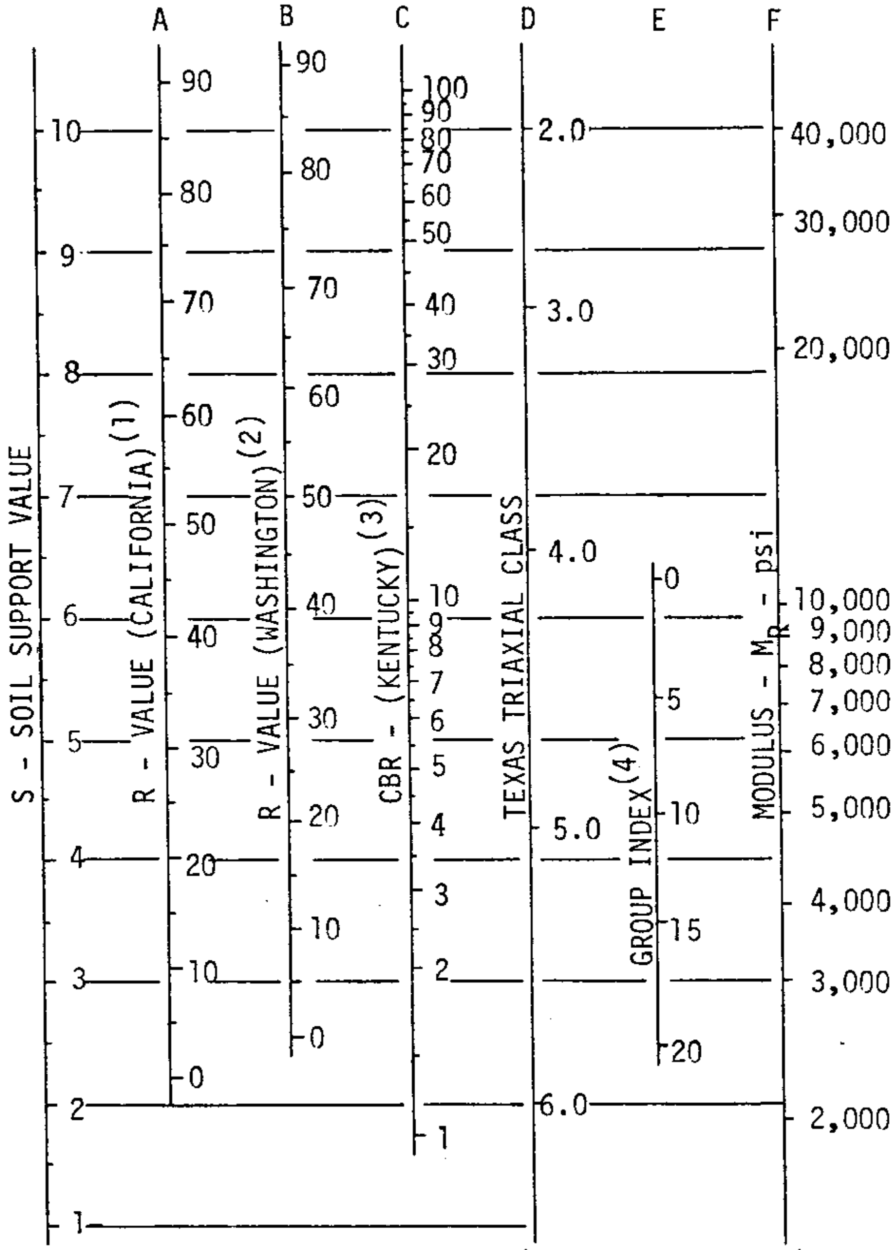
Table 4

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

Water Content (%)	Dry Density (lb./cu.ft.)	CBR (%)	Swell (%)	Water Content After Soaking (%)
		<u>SERIES 1</u>		
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
		<u>SERIES 2</u>		
13.0	114.0	13.5	1.5	16.4
16.4	112.5	7.5	0.5	17.3
19.3	106.6	2.2	0.4	19.6

NOTE: Specimens soaked 4 days, 10 lb. surcharge weight.

<u>Series 1 compaction</u>: 10 lb. hammer, 18 in. drop, 5 layers, 12 blows per layer (12,200 ft.-lb./ft.)



(NCHRP Report 128)

(1) The correlation is with the design curves used by California; AASHO designation is T-173-60, and exudation pressure is 240 psi. See Hveem, R.M., and Carmany, R.M., "The Factors Underlying the Rational Design of Pavements." Proc. HRB, Vol. 28 (1948) pp. 101-136.

(2) The correlation is with the design curves used by Washington Dept. of Highways; exudation pressure is 300 psi. See "Flexible Pavement Design Correlation Study." HRB Bull. 133 (1956).

(3) The correlation is with the CBR design curves developed by Kentucky. See Drake, W.B., and Havens, J.H., "Re-Evaluation of Kentucky Flexible Pavement Design Criterion." HRB Bull. 233 (1959) pp. 33-56. The following conditions apply to the laboratory-modified CBR: specimen is to be molded at or near the optimum moisture content as determined by AASHO T-99; dynamic compaction is to be used with a hammer weight of 10 lb dropped from a height of 18 in.; specimen is to be compacted in five equal layers with each layer receiving 10 blows; specimen is to be soaked for 4 days.

(4) This scale has been developed by comparison between the California R-value and the Group Index determined by the procedure in Proc. HRB Vol. 25 (1945) pp. 376-392.

Figure 4: Correlation chart for Estimating Soil Support (S).

Table 5

COMPARISON OF STRENGTH TESTS FOR
PALOUSE SILT

Moisture Content %	Strength Tests				
	Modulus[1] Md x 10^3	CBR		R-value	
		Lab[2]	Calculated[3]	Lab[4]	Calculated[3]
13.0	30	13.5	60		
16.4	14	7.5	20		
19.3	3.6	2.2	2.5		
21.7	3			31	5
22.7	2.9			16	4
23.3	2.8			15	4
24.0	2.8			10	4

1. From tests conducted by Kingham and Kallas [7]
2. See Table 4.
3. See Figure 4.
4. See Table 3.

Figure 5: Schematic Profile for Ring #1

Sections 1 & 4: Cement Treated Base (Screened Aggregate)
Sections 2 & 5: Class "E" Asphalt Concrete Base
Sections 3 & 6: Asphalt Treated Base (Screened Aggregate)

4.25" Class "B" A.C. Surfacing		
#1 - 7.75" Cement Treated Base (Screened Aggregate)	#2 - 5.25" Asphalt Treated Base	#3 - 5.25" Class "E" A.C.
6.0" Crushed Rock	8.5" Crushed Rock	8.5" Crushed Rock

A - 6 (10) Soil

4.25" Class "B" A.C. Surfacing		
#4 - 7.75" Cement Treated Base (Screened Aggregate)	#5 - 5.25" Asphalt Treated Base	#6 - 5.25" Class "E" A.C.
6.0" Crushed Rock	8.5" Crushed Rock	8.5" Crushed Rock

A - 6 (10) Soil

the three saturated sections was the C.T.B., the Class "E" A.C. base, and then the A.T.B., at 2.44 m, 2.85 m, and 3.15 m wheel passes, respectively. The three remaining sections developed severe rutting, and the test was stopped after 4,724,100 wheel passes. The only sign of distress was surface rutting. It was discovered that the CTB was fatigue cracked when the three surviving sections were uncovered.

The only measuring devices used were hydraulic pressure cells. Readings were less than could reasonably be expected. Difficulties occurred with their operation. No other measurements were made except for the measurement of surfacing rutting. This lack of instrumentation resulted in little information on the behavior of these pavements other than their wear characteristics. More detailed findings are presented in reference 1.

The structural sections in Rings 2, 3, and 4 were reduced, in the hope that the sections would fail between several hundred to 2 million 10,000-pound wheel loads. The control base type, which was the thickest crushed surfacing top course, was planned to fail somewhere between 500,000 and 1,000,000 load applications of a 10-kip wheel load on a subgrade soil with an effective "R" value of 40-50.

Figures 6, 7, and 8 show the sections constructed. The thicknesses selected were successfully tested to fail at about 10,000 to 800,000 wheel loads. A more thorough discussion of the rings by Krukar and Cook is presented in references 2, 3, and 4.

MIX DESIGN

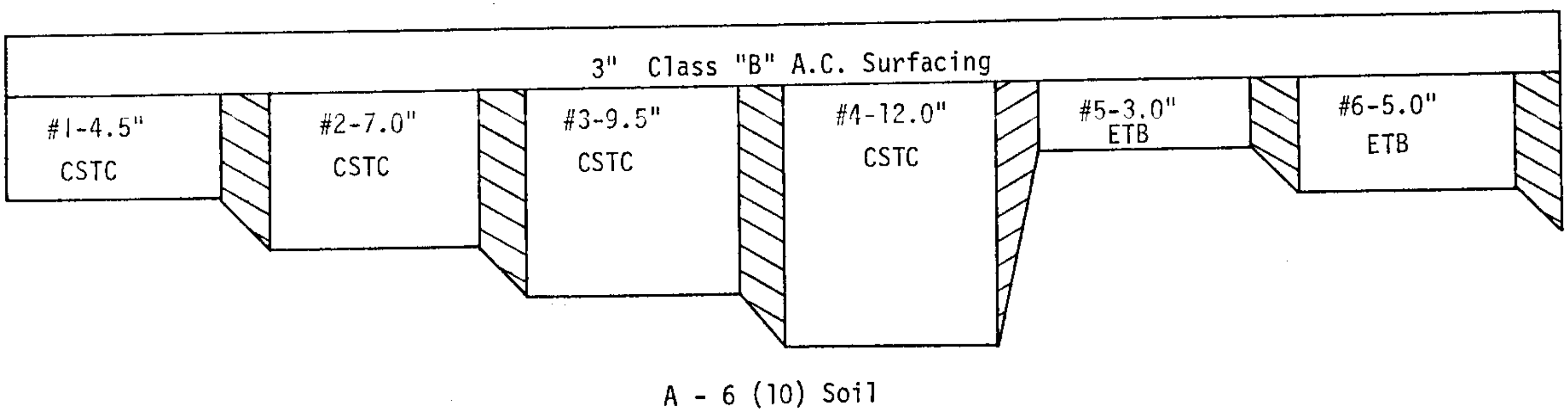
Washington State Materials Laboratory provided the mix designs for the various asphaltic mixes used in the test sections. Tables 6, 7, and 8 list the mix design requirements and average of mixes used. Some difficulties were encountered in obtaining a uniform mix at the specified asphalt content

Figure 6: Schematic Profile for Ring #2

Sections 1 - 4: Untreated Crushed Surfacing Topcourse

Sections 5 - 8: Emulsion Treated Crushed Surfacing Topcourse

Sections 9 - 12: Special Asphalt Treated Base -- Non-Fractured Screened Aggregate



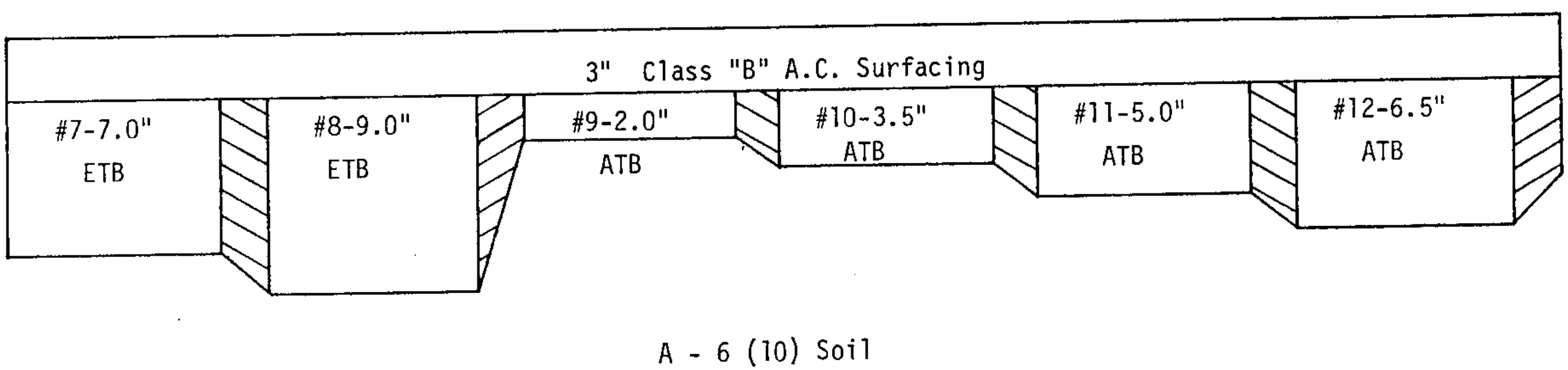
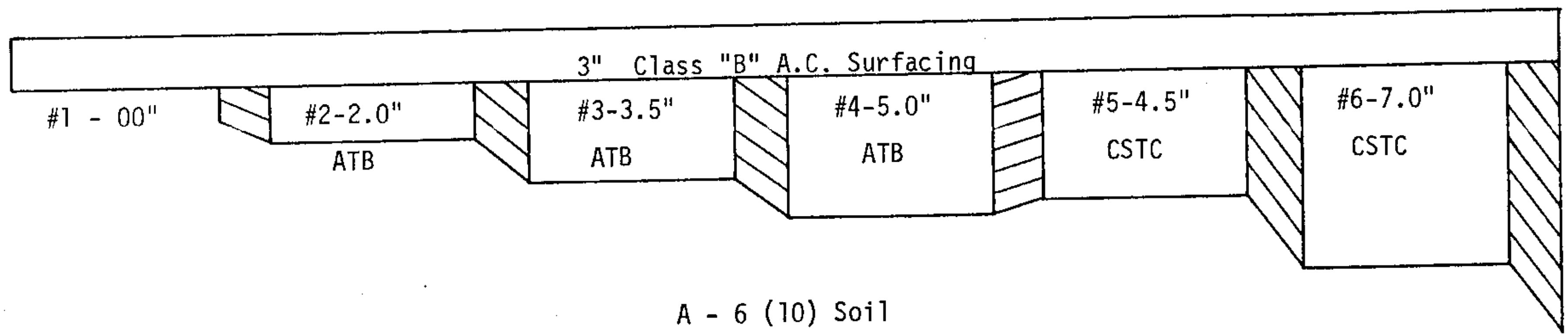


Figure 7: Schematic Profile for Ring #3

Sections 1 - 4: Special Aggregate Asphalt Treated Base (ATB)

Sections 5 - 8: Untreated Crushed Surfacing Top Course Base (UTB)

Sections 9 - 12: Emulsion Treated Crushed Surfacing Top Course Base (ETB)



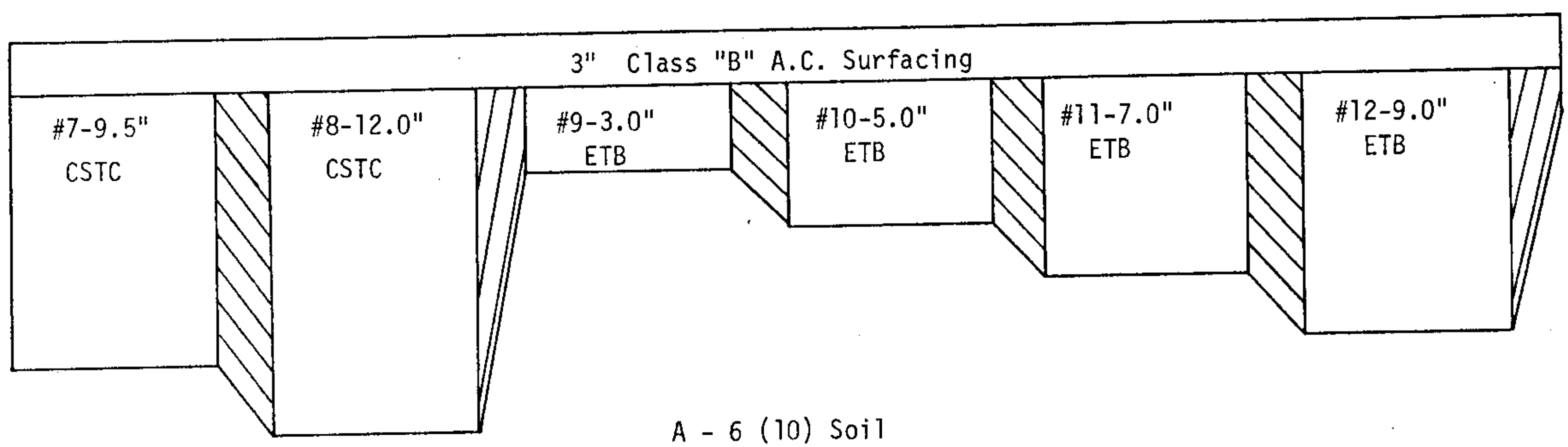


Figure 8: Schematic Profile for Ring #4

Sections 1 - 4: Sand-Asphalt Base (SAB)
Sections 5 - 8: Class "F" Asphalt Concrete Base (ACB)
Sections 9 - 12: Untreated Crushed Surfacing Topcourse Base (UTB)

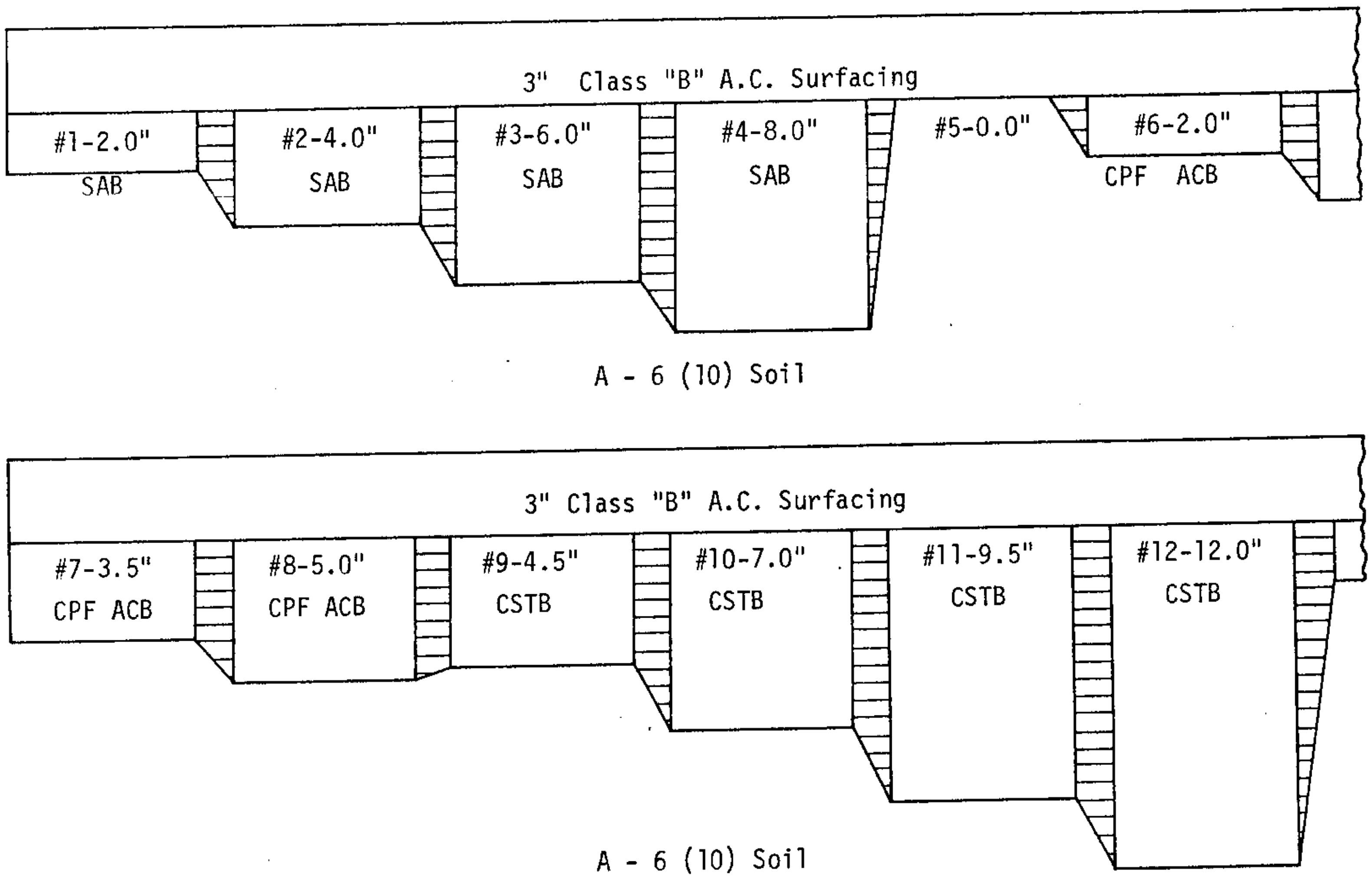


TABLE 6: MIX DESIGN REQUIREMENTS, Ring 2

Sieves-% Passing	Emulsion-Treated Base (C.S.T.C.)		Special Asphalt-Treated Base (Non-Fractured)		Cl. "B" A.C. Wearing Course	
	Specified	From * Extraction	Specified	From * Extraction	Specified	From * Extraction
2" Square			100	100		
1" Square						
5/8" Square	100	100		100	100	100
1/2" Square		99	56-90	90	90-100	97
1/4" Square	50-65	64	40-78	59	55-75	65
U.S. No. 10		30	22-52	44	32-48	39
U.S. No. 40	8-23	19	8-32	21	11-24	20
U.S. No. 80		15		10	6-15	13
U.S. No. 200	10 max.	10	2-9	6	3-7	8
Sand Equivalent % Min.	40		35			
% Fractures			5% max.	3% max.	75% min.	
Penetration Grade A.C.	SS-Kh		60-70	60-70	85-100	85-100
Amount of A.C. %	5.0 Emulsion	3.5 Residue	2.8	3.0	5.3	5.2
Stabilometer "S" Valves	30	32	20	28	30	29
Cohesiometer "C" Valve	100	200	50	158	100	260
Modified Immersion Compression (MIC)Test-% Ret.	70		70		70	
Wt. Per Cu. Ft. of Mix	140.4	148.3**	148.5	145.8	152.0	158.8
Voids-Volume in Mix						3.6
Rice Density						162.3

* Extracted and tested by the Washington Highway Department, Materials Laboratory at Olympia, Washington

** Compacted at room temperature, tested at 140^{o}F.

TABLE 7: MIX DESIGN REQUIREMENTS, Ring 3

Sieves-% Passing	Emulsion Treated Base (C.S.T.C.)		Special Asphalt Treated Base (Non-Fractured)		Cl. "B" A.C. Wearing Course	
	Specified	From * Extraction	Specified	From * Extraction	Specified	From * Extraction
2" Square			100	100		
1" Square				100		
5/8" Square	100	100		95	100	100
1/2" Square		99	56-90	89	90-100	100
1/4" Square	50-65	83	40-78	67	55-75	73
U.S. No. 10		59	22-52	41	32-48	42
U.S. No. 40	8-23	25	8-32	20	11-24	23
U.S. No. 80		15		11	6-15	14
U.S. No. 200	10 max.	9.2	2-9	7.6	3-7	9
Sand Equivalent % Min.	40		35			
% Fractures			5% max.	3%	75% min.	
Penetration Grade A.C.	SS-Kh		60-70	60-70	85-100	60-70
Amount of A.C. %	5.0 Emulsion	2.9 Residue	2.8	3.0	5.3	5.7
Wt. Per Cu. Ft. of Mix	140.4	133.5	148.5	143.2	152.0	149.4
Voids-Volume in Mix		17.0				1,2
Rice Density		2.579				2.580

* Extracted and tested by the Washington Highway Department, Materials Laboratory at Olympia, Washington.

Samples submitted at the end of testing of Ring 3.

TABLE 8: MIX DESIGN REQUIREMENTS, Ring 4

Sieves--% Passing	Untreated Base (C.S.T.C.)		Sand-Asphalt Base (S.A.B.)		Asphalt Concrete Base Cl "F"		Asphalt Concrete Cl "B" Pavement	
	Specified	In Place	Specified	From Extraction *	Specified	From Extraction *	Specified	From Extraction *
2" Square								
1" Square								
3/4" Square			100	100	100	100		
5/8" Square	100	100					100	100
1/2" Square					80-100	93	90-100	100
1/4" Square	50-65	60	70-100	100	45-75	59	55-75	70
U.S. No. 4		50						
U.S. No. 10		32	55-100	100	30-50	36	32-48	44
U.S. No. 40	5-23	13	20-100	33			11-24	25
U.S. No. 80						9	6-15	14
U.S. No. 100		7		9				13
U.S. No. 200	10 max.	4	0-20	3	2-8	5	3-7	7
Sand Equivalent % Min.	40		30		45		45	
% Fractures	75	100			50	72	75	100
Penetration Grade A.C.			85-100	85-100	85-100	85-100	85-100	85-100
Amount of A.C. %			5.0	5.3	5.5	5.8	5.3	6.1
Wt. Per Cu. Ft. of Mix	137.5	142.3	128.1	113.5	156.0	146.6	156.0	147.5

* Extracted and tested at Washington Highway Department, Materials Laboratory at Olympia, Washington.

for several of the mixes. Some of the construction and quality control problems are mentioned in references 2, 3, and 4. For instance, Table 8 lists the design asphalt content for Class B asphalt concrete as 5.3%. The five field samples used to determine the field mix had asphalt contents of 5.3%, 6.0%, 6.5%, 6.5%, and 6.2%, for an average of 6.1% as shown. The five extracted samples of Class F asphalt concrete base for Ring 4 had asphalt contents of 6.4%, 6.4%, 5.1%, 5.5%, and 5.7%, for an average of 5.8%. The design mix for the sand asphalt base was 5.0% asphalt cement, but the extracted samples showed 5.2%, 5.3%, 5.3%, 5.5%, and 5.4%, for an average of 5.3%. Finally, the Class B asphalt concrete used in Ring 3 (Table 7) had an average asphalt cement content of 5.7%, 0.4% over the design mix. These differences, which make it appear as if there was an apparent lack of quality control, make it very difficult to conduct a proper analysis of the performance of the pavements. The possibility of this variation exists in the real world but is probably not being detected. The test track personnel tested for more samples than are usually required for quality control for the quantity of materials involved. Materials containing an excess of asphalt are probably less stable and, therefore, are more susceptible to rutting and shoving than the drier mixes. Alternately, the rich mixes will have greater fatigue properties than the dry mixes. Because of these variations, only general conclusions can be made about the performance of the sections.

PREDICTION MODELS OF PAVEMENT PERFORMANCE

BRIEF DISCUSSION OF TEST TRACK RESULTS

A complete description of the performance of the three test rings is contained in references 2, 3, 4, and 6. Although it is not the intent of this report to duplicate this information, some information is contained herein for ease of explanation. Tables 9, 10, and 11 list the number of wheel loads to a given distress state for each section. The Benkelman beam deflection and rut depth measured at each distress state is also listed. Some general observations can be made by comparing the performance of similar sections. The untreated base sections, common in all three test rings, showed a considerable variation in performance. The life of the UTB to first cracking for Ring 2 was from 210,300 to 232,600 wheel loads; Ring 3 from 679,100 to 870,600; and Ring 4 from 36,600 to 144,700. The differences can be attributed to factors such as material, construction, and climatic condition variations. Although the asphalt content of the Class B wearing course varied between rings, it was not consistent with the different life spans. That is, the asphalt content of Ring 2 averaged 5.2% and lasted about 220,000 wheel loads to first cracking; Ring 3 averaged 5.7% and lasted about 750,000 wheel loads. However, Ring 4 had the highest asphalt content of 6.1% and lasted only about 100,000 wheel loads. Kasianchuk [12] found that an optimum asphalt content existed in terms of fatigue life. Figure 9 shows the results found for a mix used on Ygnacia Valley Road. This report indicates that an asphalt content of 6.8% provided the greatest fatigue life. It is possible that the asphalt content of 6.1% used in Ring 4 exceeded this optimum point for fatigue and, therefore, was the cause of the short life span. More likely, the environmental conditions contributed more to the rapid failure of the thin sections than the asphalt content. Using a higher asphalt content may increase fatigue life at the expense of stability and possible bleeding. However, fatigue tests for this particular material at different asphalt contents are not available to confirm this possibility.

Construction and climatic variations may also have caused the differences in life span. Tables 12, 13, and 14 list the initial densities and moisture contents of the subgrade. More importantly, Figures 10 through 18 show the

Table 9

Table 9: Critical B.B. Deflections(B.B.D.),** Rut Depths(R.D.), and Wheel Passes(W.P.)--Ring No.

Type	Section	Initial Distress			Alligator Cracking			Ultimate Failure		
		W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.D
J.T.B.	1	330.12[1] 157.2	0.050[2] 0.053	0.25	367.5 175.0	0.047 0.055	0.25	441.63 210.3	0.155 0.180	2.5
	2	207.9 99.0	0.107 0.125	0.25	273.0 130.0	0.123 0.144	0.25	431.34 205.4	0.137 0.160	2.5
	3	210.0 100.0	0.062 0.073	0.25	273.0 130.00	0.066 0.077	0.25	434.07 206.7	0.104 0.120	0.7
	4	332.14 153.4	0.049 0.058	0.20	---	---	---	488.46 232.6	0.042 0.050	1.0
A.T.B.	9	437.43 208.3	0.211 0.246	0.25	---*	---	---	438.69 208.9	0.211 0.246	2.5
	10	443.73 211.3	0.142 0.166	0.75	---*	---	---	464.94 221.4	0.120 0.140	2.5
	11	453.6 216.0	0.103 0.119	0.50	---*	---	---	488.46 232.6	0.148 0.172	2.0
	12	485.52 231.2	0.077 0.090	0.75	---*	---	---	488.46 232.6	0.079 0.092	1.0
E.T.B.	5	207.9 99.0	0.077 0.090	0.25	252.0 120.0	0.082 0.095	0.50	418.32 199.2	0.086 0.100	1.25
	6	207.9 99.0	0.090 0.104	0.25	252.0 120.0	0.090 0.104	0.50	418.32 199.2	0.097 0.112	1.25
	7	368.76 175.6	0.045 0.053	0.20	420.0 200.0	0.046 0.054	0.22	484.89 230.9	0.167 0.195	1.25
	8	415.38 197.8	0.038 0.045	0.20	---	---	---	488.46 232.6	0.090 0.104	1.00

* Did not exhibit alligator cracking pattern failure

** Not corrected for temperature

1 Corrected into equivalent 18-kip single axle loads

2 Corrected to single axle 18-kip load

Table 10

Table 10: Critical B.B. Deflections(B.B.D.), Rut Depths(R.D.), and Wheel Passes(W.P.)--Ring No.3

Material	Section	Initial Distress			Alligator Cracking			Ultimate Failure		
		W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.D.
I.T.B.	5	795.9[1] 379.0	0.036[2] 0.043[3]	0.25	1092.0 520.	0.038 0.045	0.71	1426.11 679.1	0.068 0.080	1.25
	6	795.9 379.0	0.053 0.062	0.25	1092.0 520.	0.075 0.087	0.71	1426.11 679.1	0.099 0.115	1.00
	7	1336.02 636.2	0.051 0.060	0.50	1428.0 680.	0.058 0.068	1.00	1544.76 735.6	0.086 0.100	1.50
	8	1527.12 727.2	0.055 0.065	0.55	1596.0 760.	0.040 0.047	0.70	1828.26 870.6	0.038 0.045	1.25
A.T.B.	1	795.9 379.0	0.047 0.055	0.15	1092.0 520.	0.049 0.058	0.022	1426.11 679.1	0.047 0.055	1.75
	2	1406.16 669.6	0.044 0.052	0.25	1544.76 735.6	0.047 0.055	0.050	1605.24 764.4	0.060 0.070	2.10
	3	1617.42 770.2	0.064 0.075	0.75	---	---	---	1645.56 783.6	---	2.50
	4	1638.0 780.0	0.042 0.050	0.72	---	---	---	1828.26 870.6	0.043 0.051	1.00
E.T.B.[4]	9	1547.07 736.7	0.107 0.125	0.50	1549.8 738.	0.189 0.220	0.90	1563.87 744.7	---	2.00
	10	1622.67 772.7	0.073 0.085	0.87	1638.0 780.	0.077 0.090	1.00	1668.45 794.5	0.091 0.105	2.00
	11	1638.0 780.0	0.051 0.060	0.70	---	---	---	1828.26 870.6	0.050 0.059	1.00
	12	1680.0 800.0	0.053 0.062	0.50	---	---	---	1828.26 870.6	0.048 0.057	0.85

1 Corrected to equivalent 18-kip single axle loads
2 Corrected for both 70° F. temperature and 18-kip single axle loads
3 Corrected only for temperature of 70° F.
4 Alligator cracking pattern did not exist as usually defined, i.e., this type of failure occurred mainly in the fall.

Table 11

Table 11: Critical B.B. Deflections(B.B.D.), Rut Depths(R.D.), and Wheel Passes(W.P.)--Ring No.

Material	Section	Initial Distress			Alligator Cracking			Ultimate Failure		
		W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.D.	W.P. x 10^3	B.B.D.	R.
U.T.B.	9	25.2[1] 12.0	0.158[2] 0.184[3]	0.25	42.0 20.0	0.172 0.200	0.50	76.86 36.6	0.189 0.220	+2.5
	10	99.54 47.4	0.139 0.162	0.60	178.5 85.0	0.185 0.215	0.70	218.82 104.2	0.215 0.250	+2.5
	11	100.8 48.0	0.156 0.182	0.50	178.5 85.0	0.163 0.190	0.70	301.14 143.4	0.215 0.250	2.0
	12	103.11 49.1	0.174 0.203	0.50	231.0 110.0	0.179 0.209	0.80	303.87 144.7	0.179 0.209	2.0
A.B.C.[4]	5	99.54 47.4	0.079 0.092	0.25	193.2 92.0	0.070 0.082	0.35	303.87 144.7	0.180 0.210	1.5
	6	312.69 148.9	0.107 0.125	0.28	319.2 152.0	0.106 0.124	0.35	332.22 158.2	0.106 0.123	+2.5
	7	338.73 161.3	0.116 0.135	0.25	357.0 170.0	0.106 0.124	0.50	359.52 171.2	0.103 0.120	2.0
	8	357.0 170.0	0.086 0.100	0.50	---	---	---	518.91 247.1	0.068 0.080	0.7
.A.B.[4]	1	303.87 144.7	0.107 0.125	0.25	319.2 152.0	---	1.00	329.7 157.0	0.107 0.125	+2.5
	2	335.58 159.8	0.133 0.155	0.80	336.0 160.0	---	1.20	346.08 164.8	0.133 0.155	+2.5
	3	368.76 175.6	0.146 0.170	0.32	---	---	---	372.75 177.5	0.113 0.132	2.0
	4	367.5 175.0	0.077 0.090	0.70	---	---	---	518.91 247.1	0.060 0.070	1.4

1 Corrected to equivalent 18-kip single axle loads

2 Corrected for both 70° F. temperature and 18-kip single axle loads

3 Corrected only for temperature of 70° F.

4 Alligator cracking pattern did not exist as usually defined, i.e., this type of failure occurred mainly in the fall.

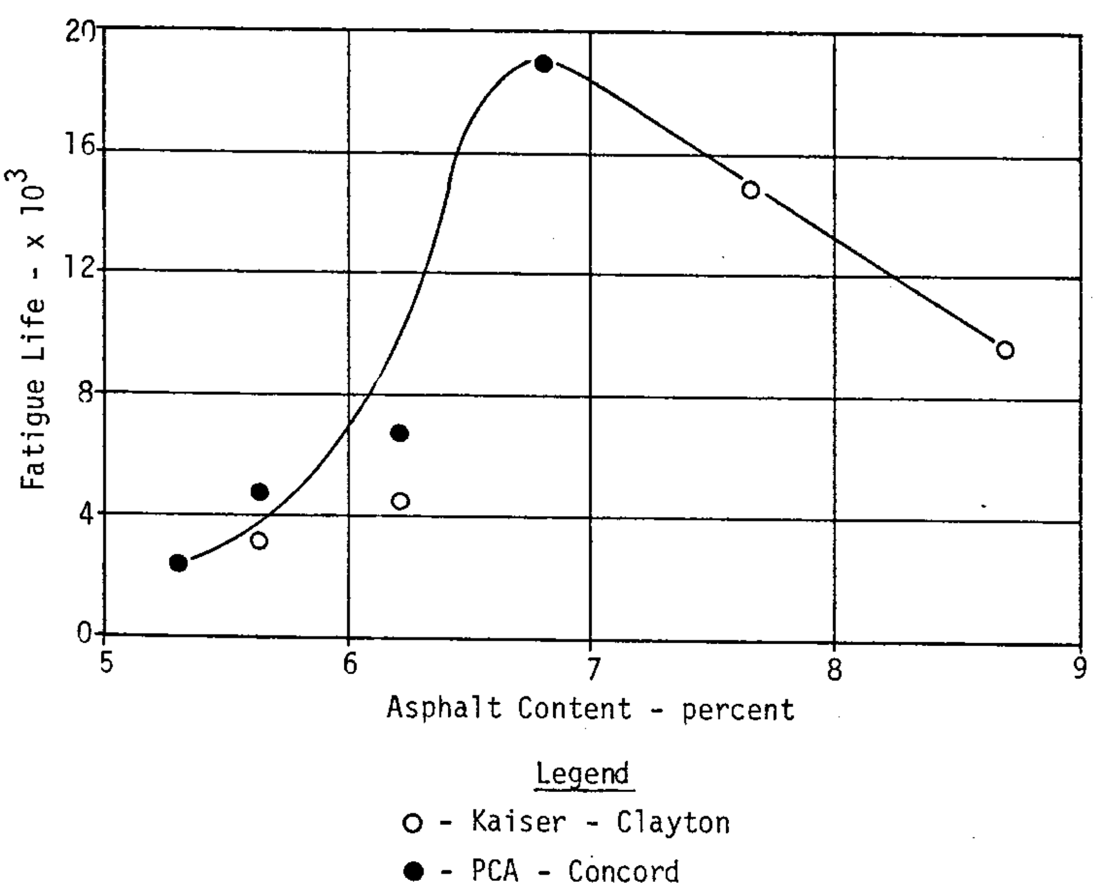


Figure 9: Results of Controlled Stress Fatigue Tests at 150-psi Tensile Stress Showing Effect of Asphalt Content, Ygnacio Valley Road.

Table 12

DENSITIES OF PALOUSE SILT SUBGRADE AT FINAL GRADE, Ring 2

Section & Type of Base	Dry Density lbs/cu. ft.	Moisture Content Per Cent	Per Cent of Maximum Density*	Per Cent of Optimum Moisture Content*
UTB 1	109.2	11.6	102.0	61.0
2	105.5	17.1	98.6	90.0
3	104.5	12.4	97.7	65.3
4	104.0	13.5	97.2	71.0
ETB 5	106.5	11.9	99.5	62.6
6	110.5	12.7	103.3	66.8
7	107.0	12.9	100.0	67.9
8	111.5	13.7	104.2	72.1
ATB 9	108.5	12.0	101.4	63.2
10	106.5	12.2	99.5	64.2
11	107.0	12.6	100.0	66.3
12	102.7	14.4	96.0	75.8
Average	107.0	13.1	100.0	69.0

Maximum Density = 107 lbs/cu. ft.   Optimum Moisture Content = 19%

*Standard Proctor Test used: ASTM - D698-57TA

Table 13

DENSITIES OF PALOUSE SILT SUBGRADE AT FINAL GRADE, Ring 3

Section & Type of Base	Dry Density lbs/cu. ft.	Moisture Content %	Per Cent of Maximum Density *	Per Cent of Optimum Moisture Content *
ATB 1	109.1	14.3	102.0	75.3
2	101.6	15.0	95.0	78.9
3	109.8	14.5	102.6	76.3
4	103.0	14.3	96.3	75.3
UTB 5	106.3	16.5	99.3	86.8
6	104.4	15.2	97.6	80.0
7	105.4	15.2	98.5	80.0
8	104.6	13.7	97.8	72.1
ETB 9	101.6	15.0	95.0	78.9
10	106.6	15.0	99.6	78.9
11	105.8	14.0	98.6	73.7
12	107.1	14.9	100.1	78.4
Average	105.4	14.8	98.5	77.9

Maximum Density = 107 lbs/cu. ft.   Optimum Moisture Content = 19%

* Standard Proctor Test used: ASTM - D698-57TA

Table 14

FINAL PALOUSE SILT SUBGRADE DENSITIES AND MOISTURE CONTENT, Ring 4

Section & Type of Base	Wet Density lb/ft^3	Dry Density lb/ft^3	Moisture Content		Percent Compaction of 95% Required Density
			lb/ft^3	%	
SAB 1	119.7	103.5	16.2	15.7	101.8
2	113.7	98.3	15.4	15.7	96.7
3	113.5	98.9	14.6	14.8	97.2
4	113.5	98.4	15.1	15.3	96.8
ACB 5	123.0	107.3	15.7	14.6	105.5
6	125.5	109.2	16.3	14.9	107.4
7	120.5	103.0	17.5	17.0	101.3
8	115.0	100.0	15.0	15.0	98.3
UTB 9	120.5	105.1	15.4	14.7	103.3
10	114.0	98.0	16.0	16.3	96.4
11	121.0	102.8	18.2	17.7	101.1
12	114.7	99.1	15.6	15.7	97.4
Mean	117.9	102.0	15.9	15.6	100.3

loss in subgrade strength throughout the life of the pavements. These curves were constructed from measuring the moisture content of the soil during the tests and using Figure 3 (page 12) to convert to subgrade stiffness, M_R. Ultimately, the subgrade stiffness of all three Rings approached 3,000 psi. However, Ring 4, the shortest lived, reached this state in less than 100,000 wheel loads, while Ring 3, the long lived, did not reach this state until about 550,000 wheel loads.

From the above discussion, it can be realized that material design, construction procedures, and climatic conditions can greatly affect the life of a pavement. Standards for material design to include gradations, asphalt content, and construction procedures have been established by the State of Washington as listed in their Standard Specifications for Road and Bridge Construction. It is imperative that these standards be followed, in order to provide a product that will perform as intended.

This points out the difficulties that any highway department encounters in maintaining construction procedures and quality control. Although the WSU Test Track materials and construction procedures were very carefully watched and checked, difficulties already mentioned were encountered. One can only speculate on how closely and uniformly the standards are checked and followed in highway construction jobs, which are much larger in scope than the WSU Test Track.

Climatic conditions cannot be controlled. However, it is possible to determine, with reasonable accuracy, the average conditions a pavement will be subjected to during its intended life. These conditions should be accounted for in any design system that is used to determine the requirements of the pavement system. An equilibrium condition is generally reached in the subgrade after the road has been in service for several months. This con-

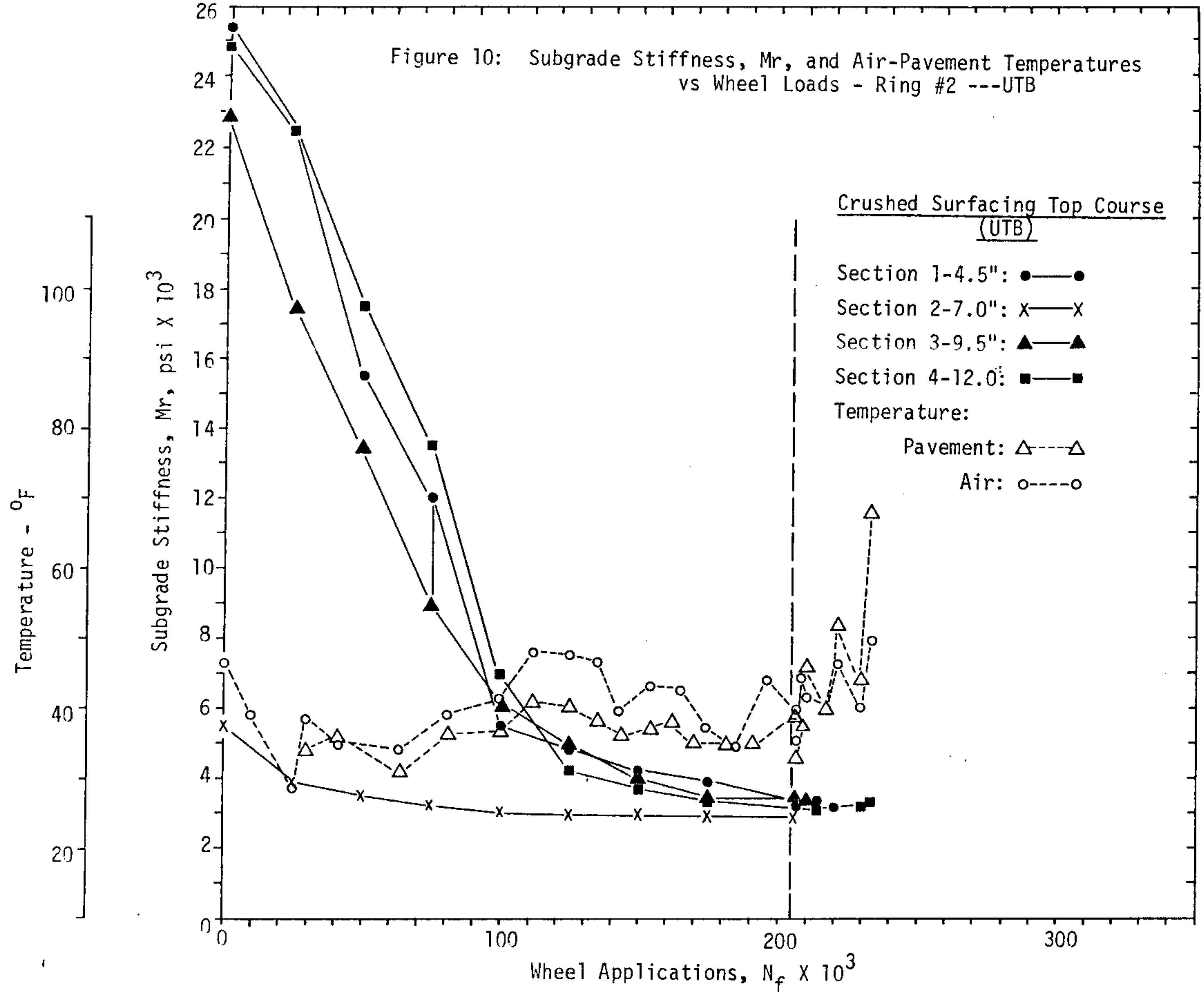
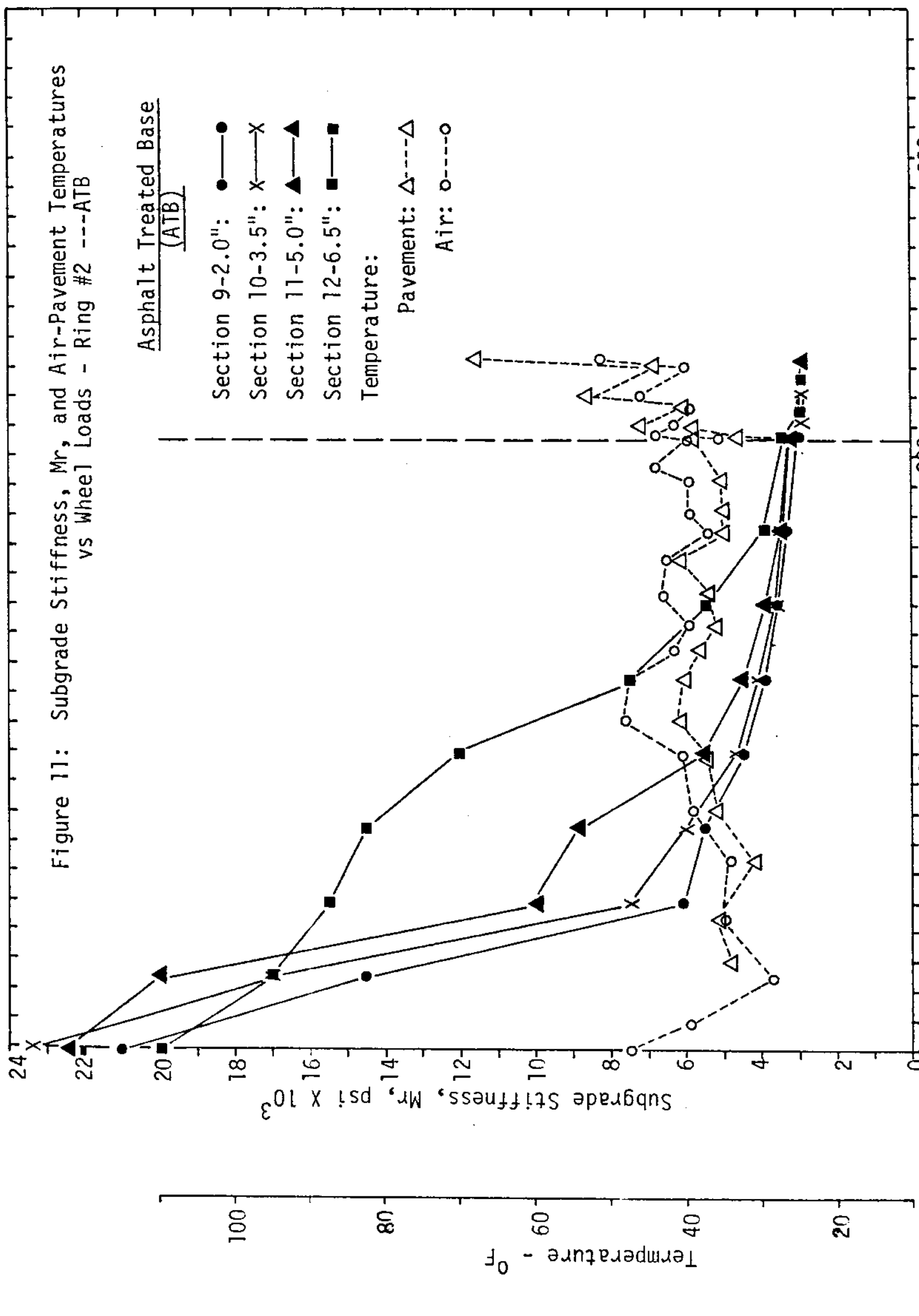


Figure 10: Subgrade Stiffness, Mr, and Air-Pavement Temperatures vs Wheel Loads - Ring #2 ---UTB

Figure 11: Subgrade Stiffness, Mr, and Air-Pavement Temperatures vs Wheel Loads - Ring #2 ---ATB



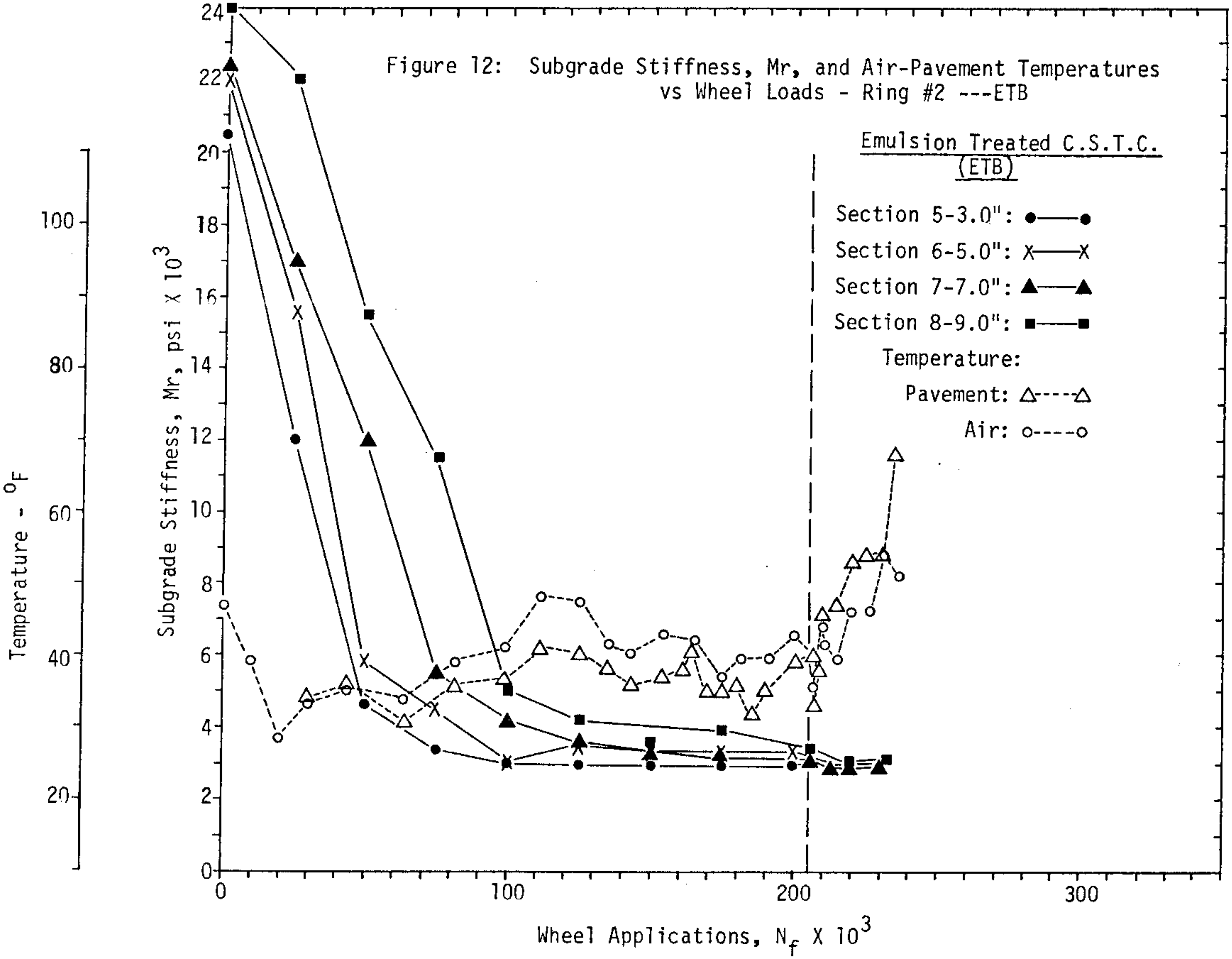


Figure 12: Subgrade Stiffness, Mr, and Air-Pavement Temperatures vs Wheel Loads - Ring #2 ---ETB

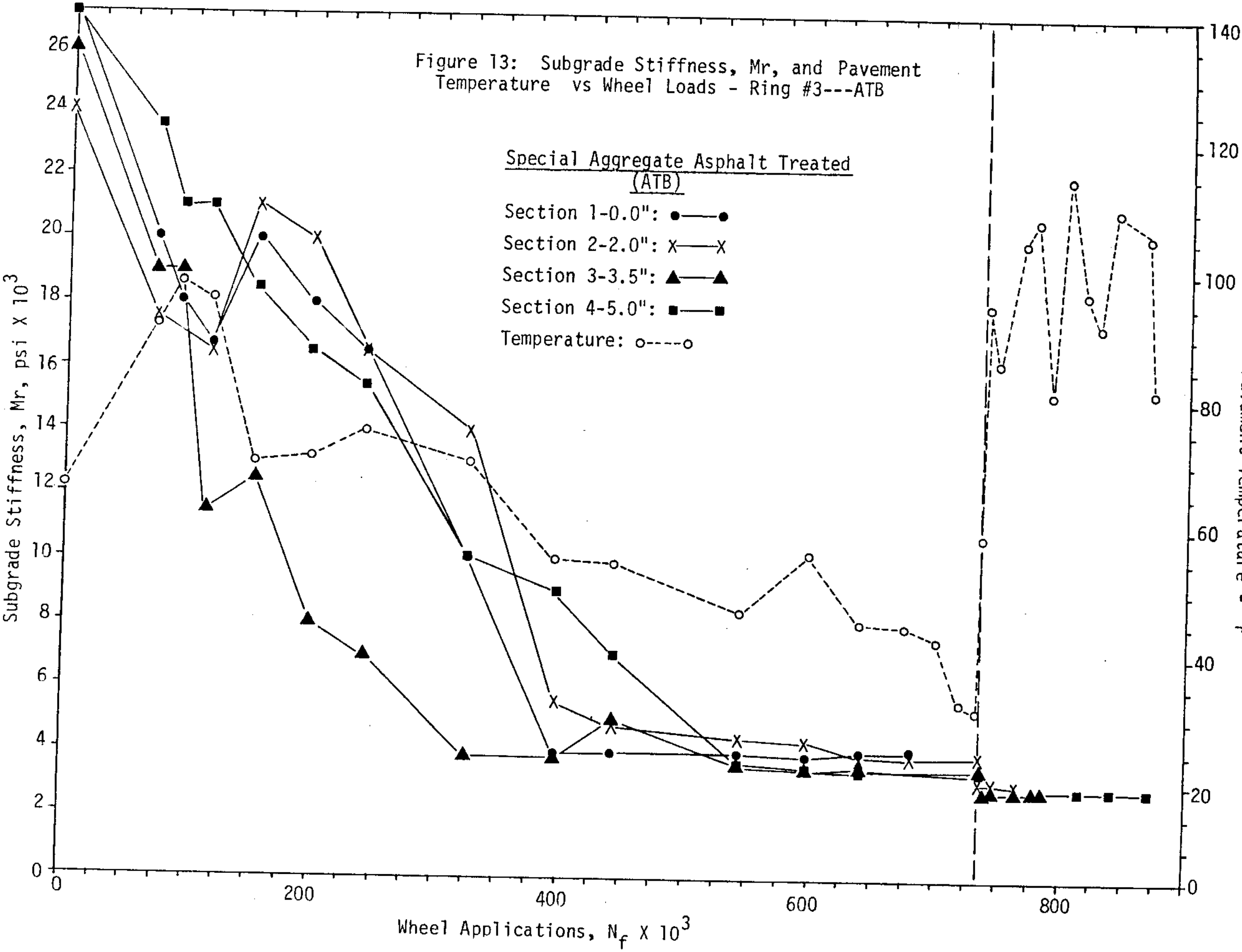


Figure 13: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #3---ATB

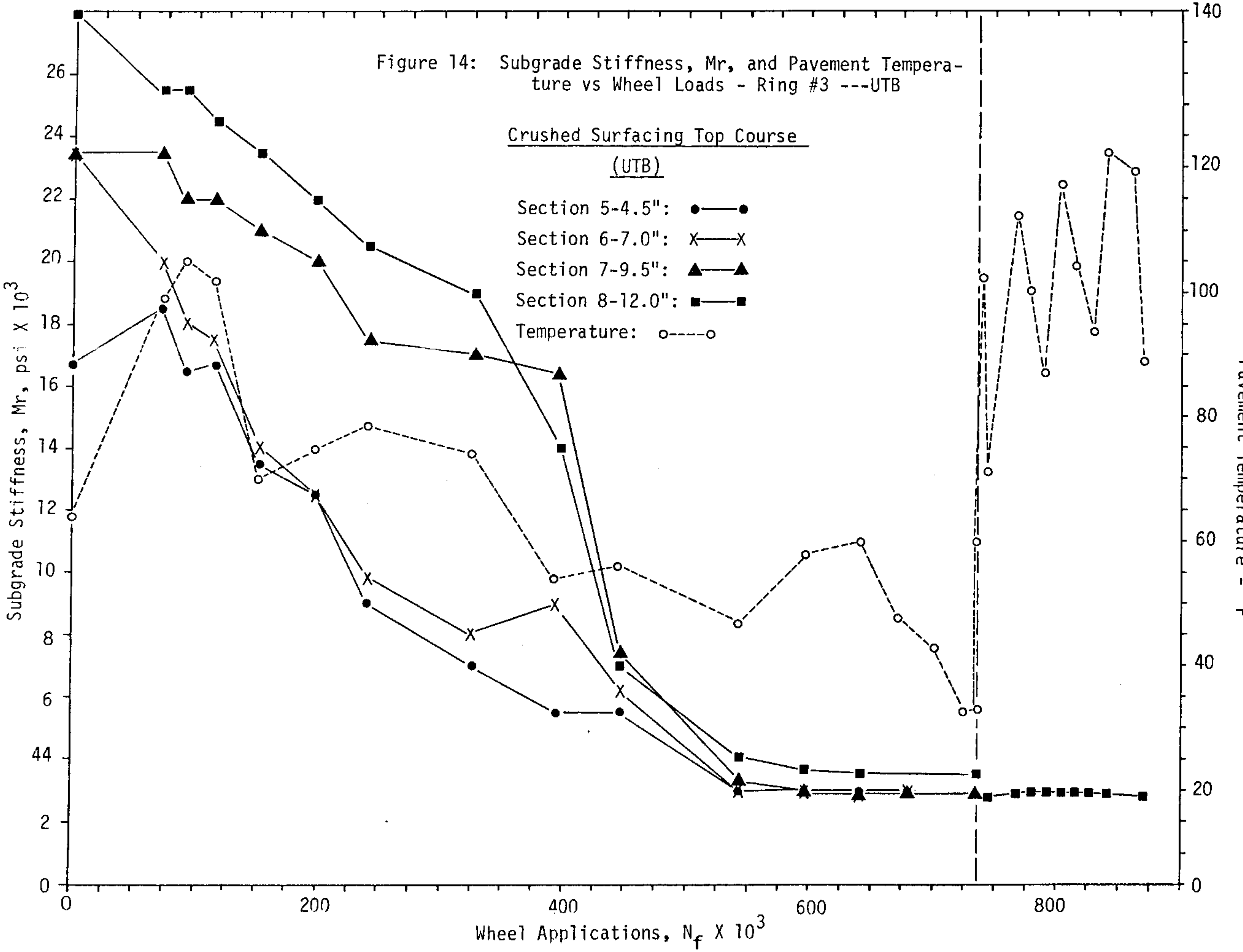


Figure 14: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #3 ---UTB

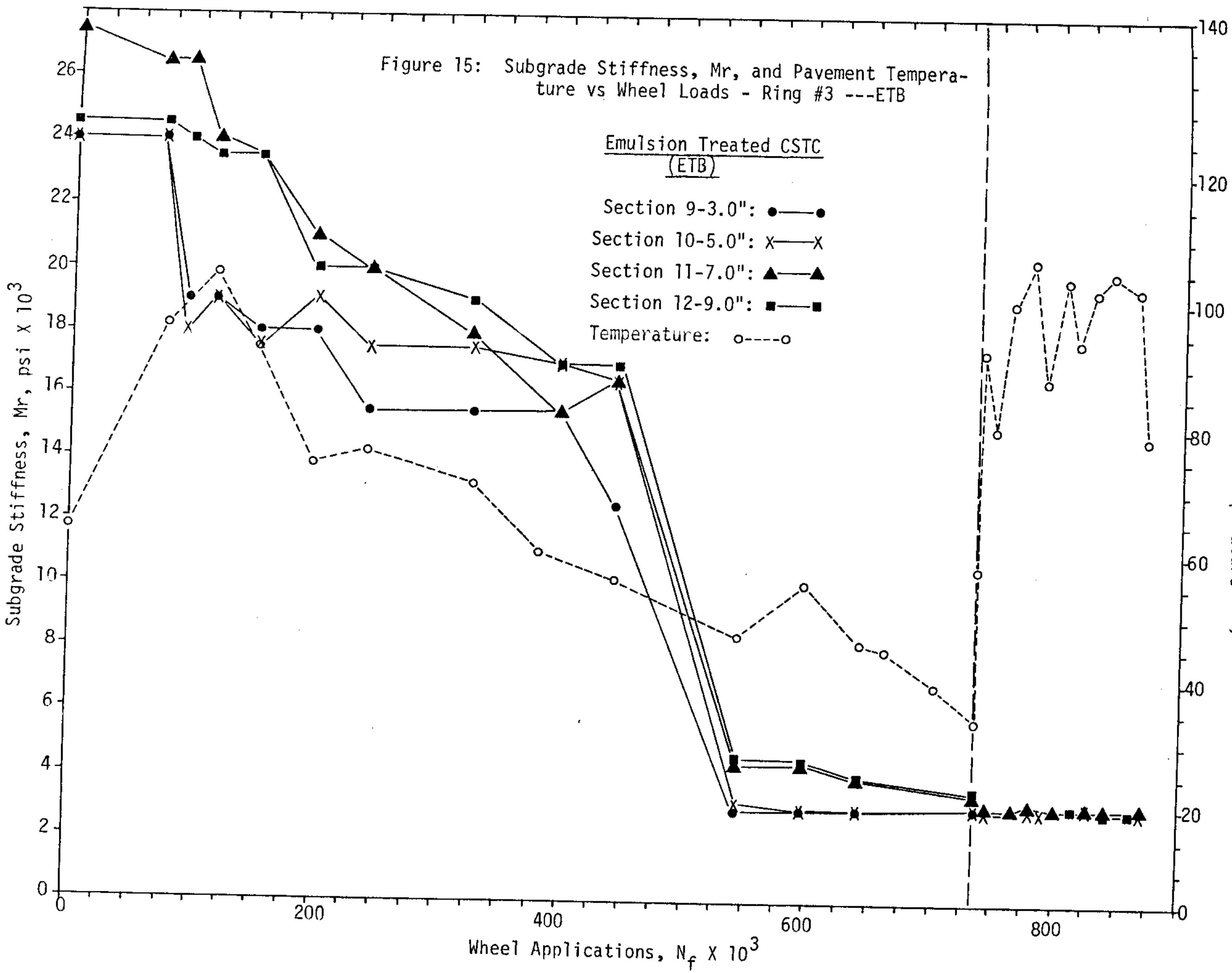


Figure 15: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #3 ---ETB

Figure 16: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #4 ---SAB

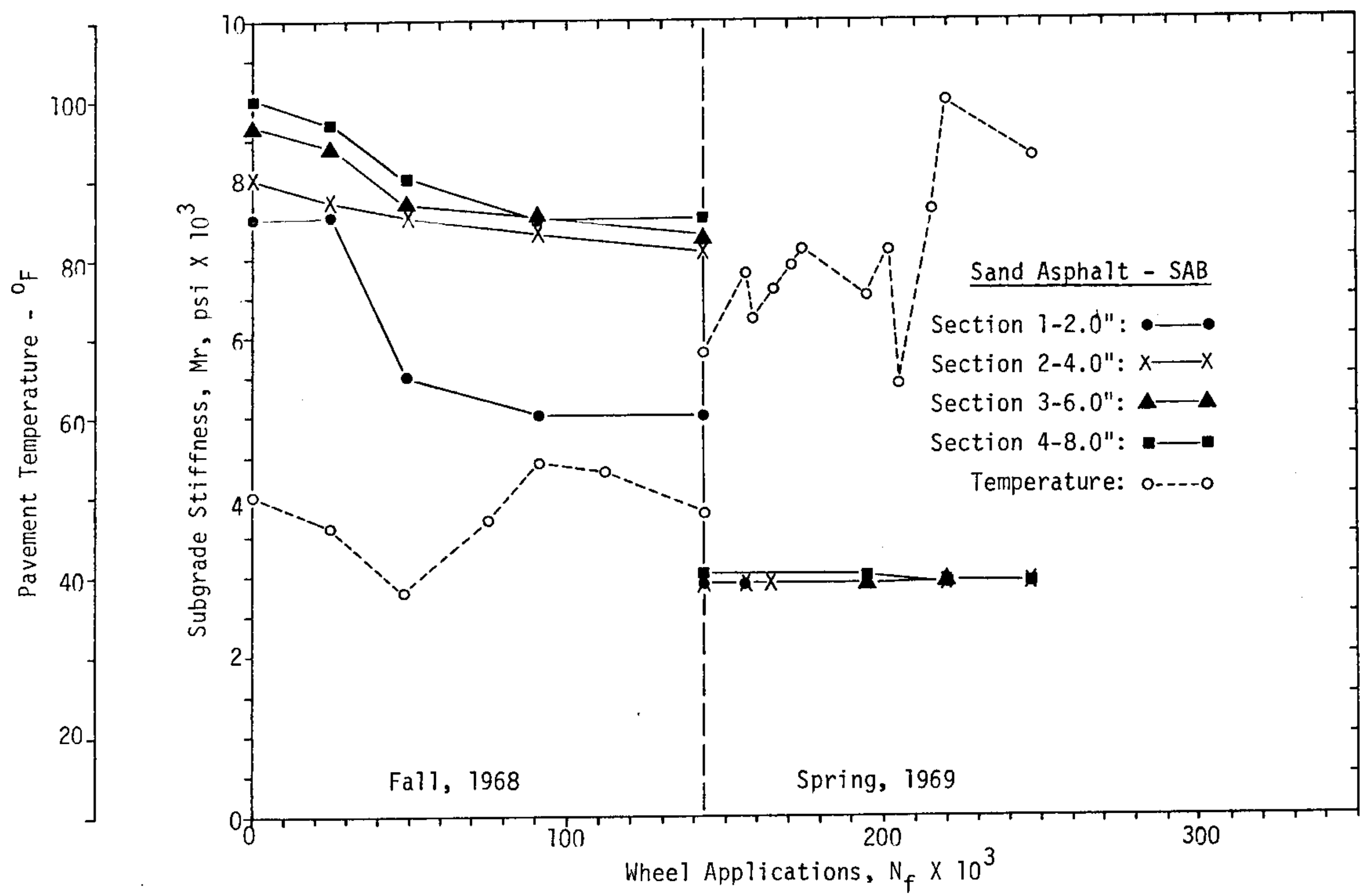


Figure 17: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #4 ---ACB

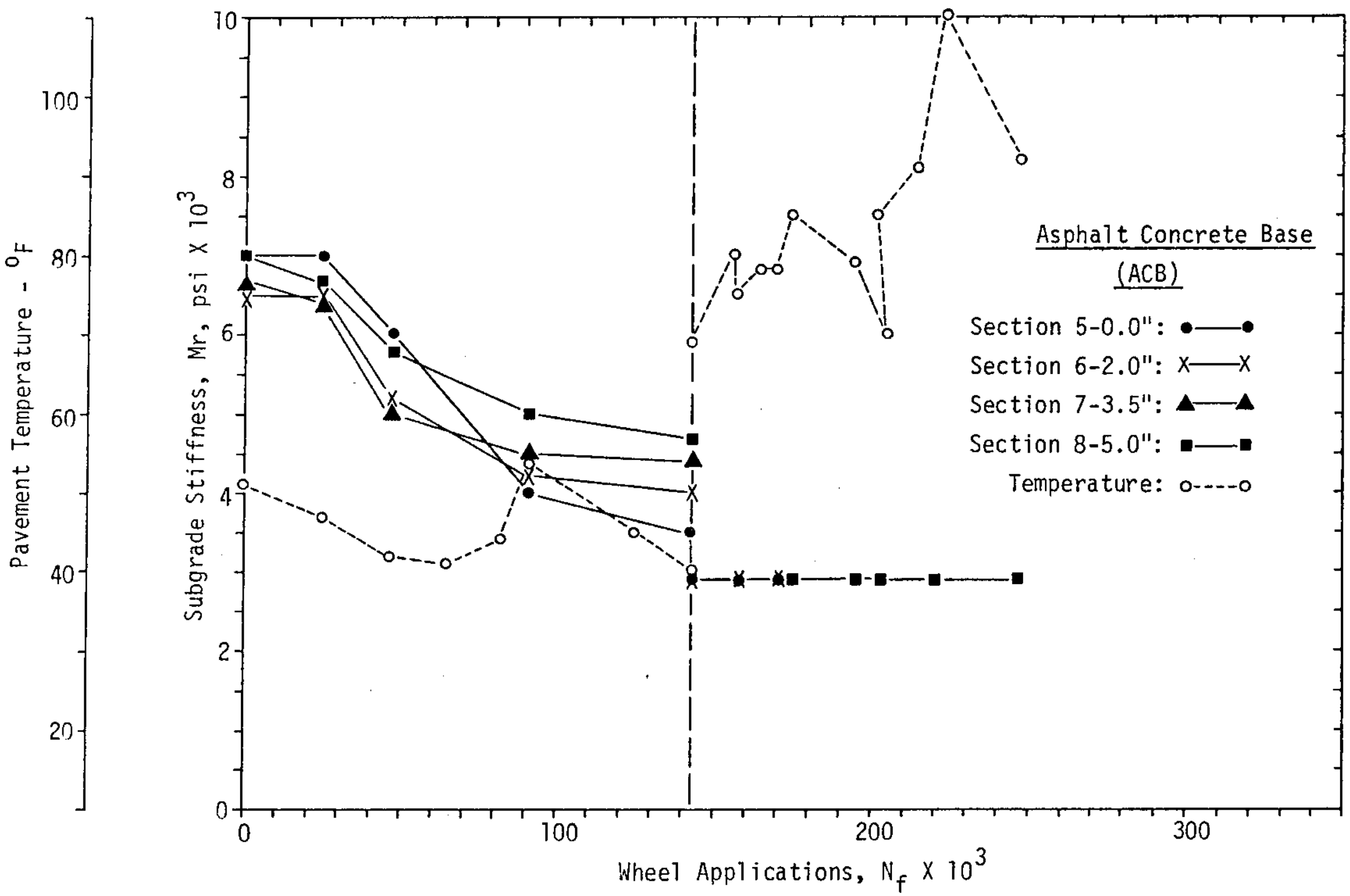
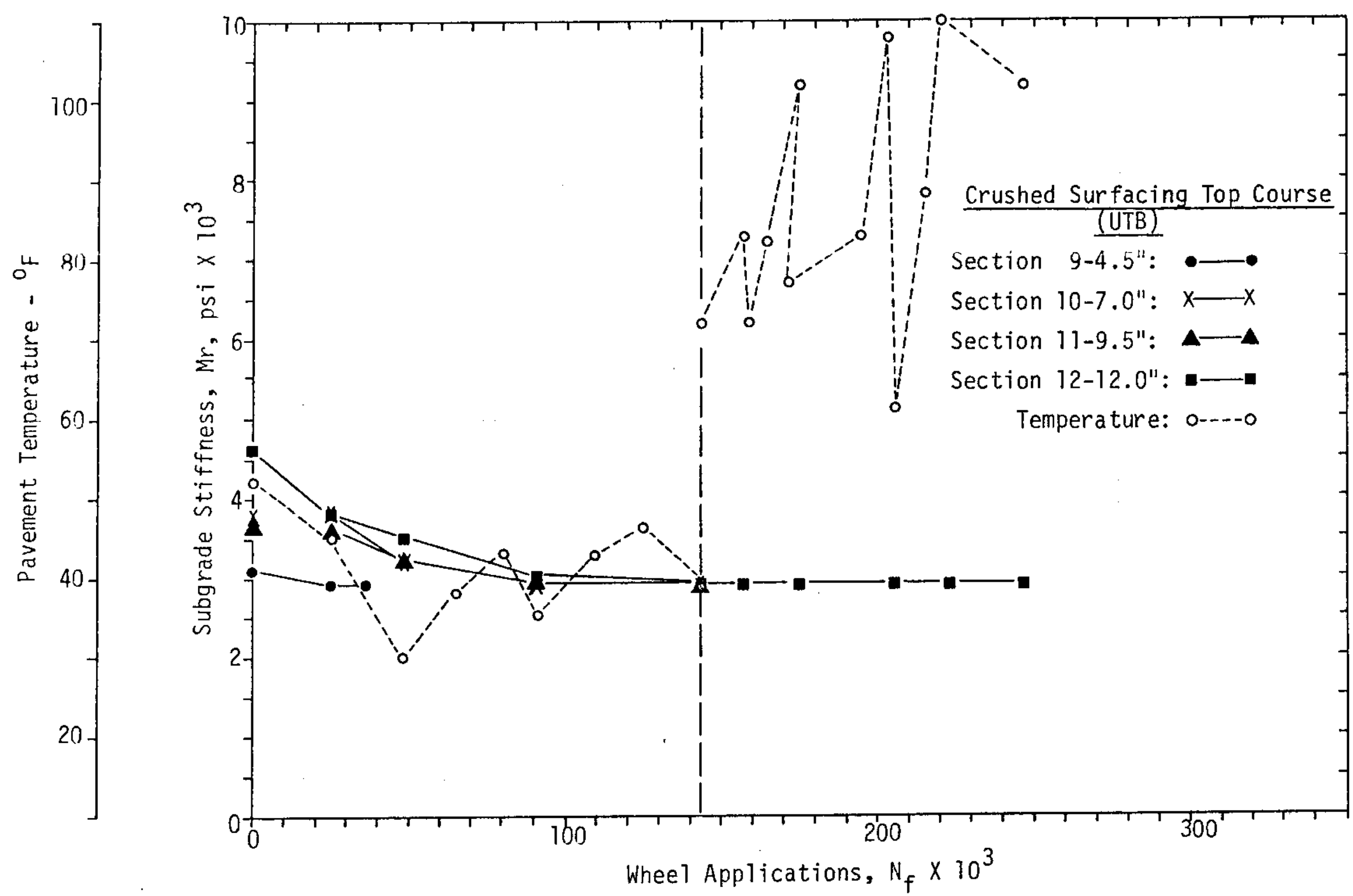


Figure 18: Subgrade Stiffness, Mr, and Pavement Temperature vs Wheel Loads - Ring #4 ---UTB



dition will not change appreciably during the life of the pavement provided adequate provisions are made to arrest the flow of free water into the pavement structure.

With the above variables taken into consideration, it is now necessary to review and select available pavement performance models that can be used to design or explain the performance of pavement systems. Two models will be examined in this report. The first, formulated by Hveem and commonly known as the R-value Design, is a semi-empirical method developed during the 1940's. It is currently being used by 10 states in the country. The second model examined is the definition of material performance in terms of its basic engineering characteristics such as modulus, stress, and strain.

R-VALUE DESIGN

Hveem and Carmany [13] presented a thorough discussion of the R-value design method in the Highway Research Board Proceedings in 1948. This paper explained the basic assumptions used in formulating the above method and described the testing procedures for the stabilometer and the expansion pressure apparatus.

Briefly, the R-value design is used to prevent excessive plastic deformation of the basement soil. This method assumes that the pavement structure will reach equilibrium conditions in relation to the state of moisture and density of the basement soil. The expansion and exudation tests are used to determine this equilibrium state. The Stabilometer is then used to determine the strength of the materials at the equilibrium state and calculate the required surfacing depth.

Hveem and Carmany also stated, "A rational solution for .. [fatigue failure] is yet to be worked out. Fortunately, however, failures .. [by fatigue failure] are somewhat in the minority and resilience of the basement soil is probably the least serious cause for distress compared to the importance of the other two considerations. It cannot be dismissed, however." [13]

No specific criterion was established to define the condition of failure for the R-value design by Hveem. Rather, subjective opinions were used to designate a pavement as adequate or inadequate. Since no previous specific criterion was available for the purpose of analysis of Rings 2, 3, and 4, the state of distress at which a pavement section has failed will be defined as that time when the pavement exhibits a maximum rut depth of 0.25 inches. This depth was selected because the individual sections seemed to deteriorate rapidly after this point was reached (see Figures 19, 20, and 21). Table 15 lists the number of wheel loads for this state of distress. It should be noted that only the maximum rut depth was recorded. A better indication of rutting damage would have been to measure both the mean and standard deviation of rut depth for the entire section. This would have helped to reduce the importance of small isolated failures caused by material or construction variations.

Another problem exists in trying to analyze the performance of the pavement sections in terms of the R-value design. The R-value design assumes the pavement structure is in equilibrium during its life. This assumption is probably correct after the first year of service. Generally, the subgrade is compacted at a lower moisture content, higher density, and, therefore, greater R-value than that used for design purposes. That is, the Proctor "Moisture-Density" Test generally selects an optimum moisture content below the equilibrium moisture content determined by the R-value design. In this case, the Proctor Test requires the Palouse silt to be compacted at 18.8% whereas the R-value equilibrium moisture content is approximately 23%. In fact, the soil was compacted below optimum moisture. Ring 2 was compacted

Figure 19: Comparison of Maximum Rut Depth and B. B. Deflection with Wheel Loads - Ring #2 ---UTB

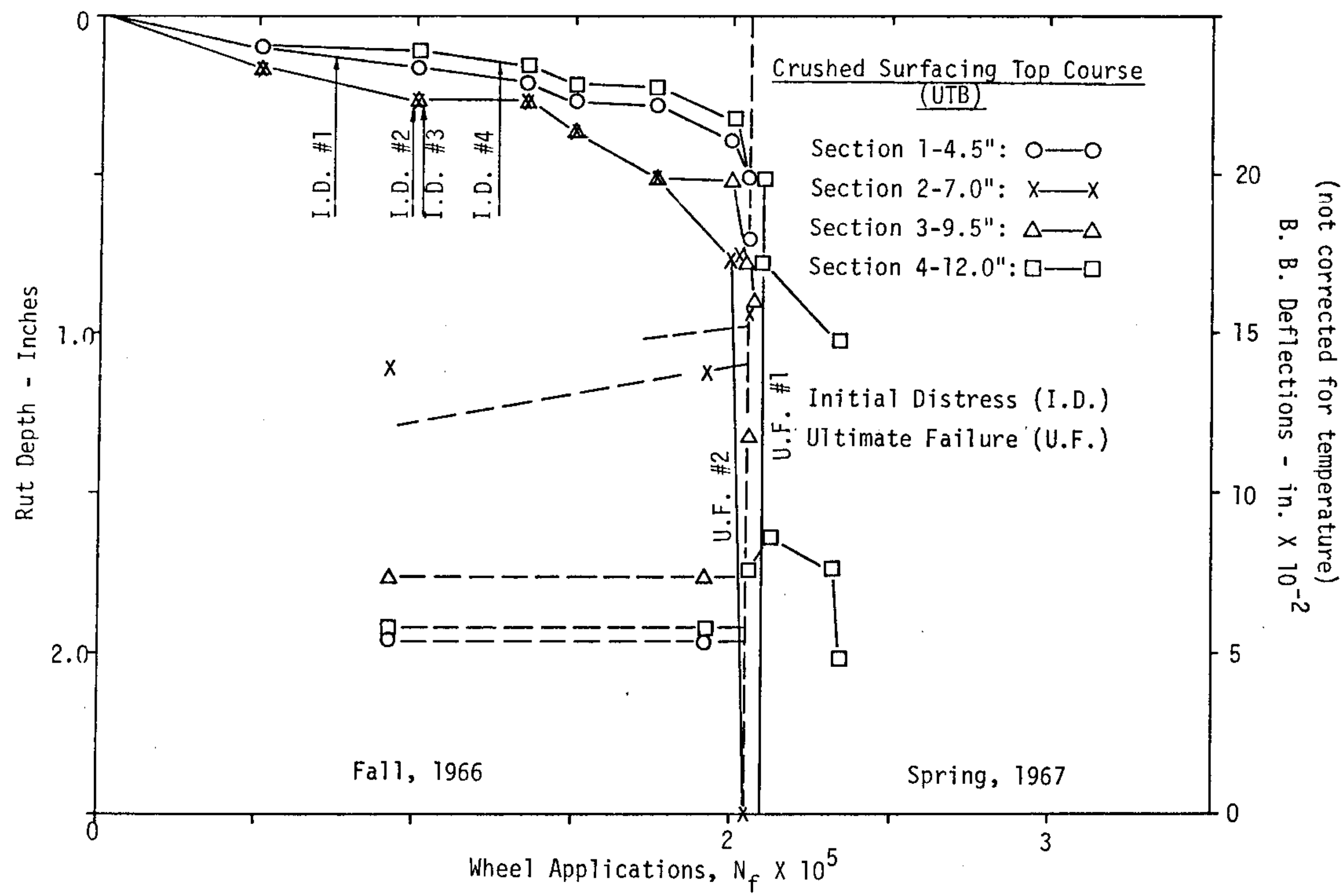


Figure 20: Comparison of Maximum Rut Depth and B. B. Deflection with Wheel Loads - Ring #3 ---UTB

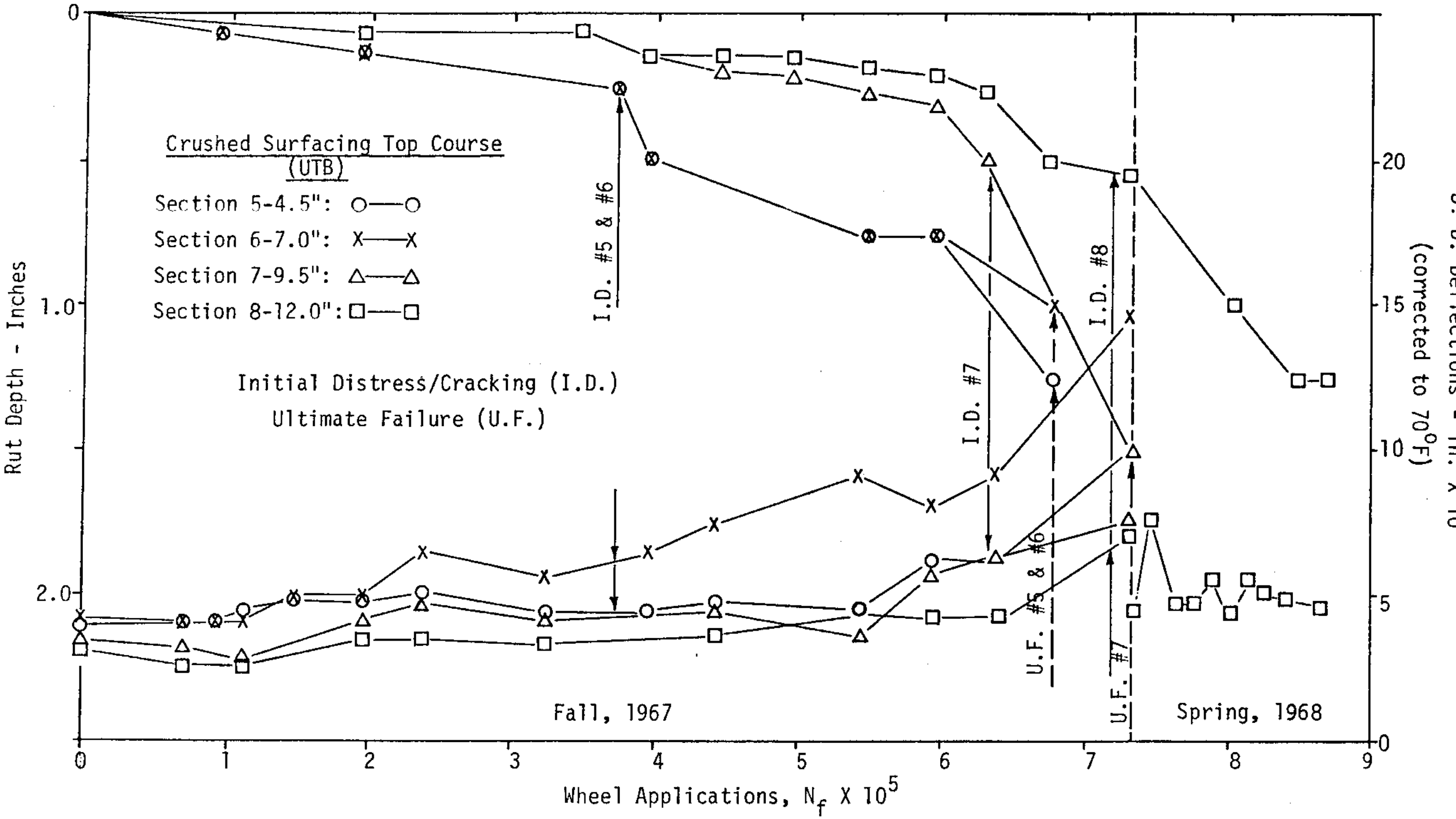


Figure 21: Comparison of Maximum Rut Depth and B. B. Deflection with Wheel Loads - Ring #4 ---UTB

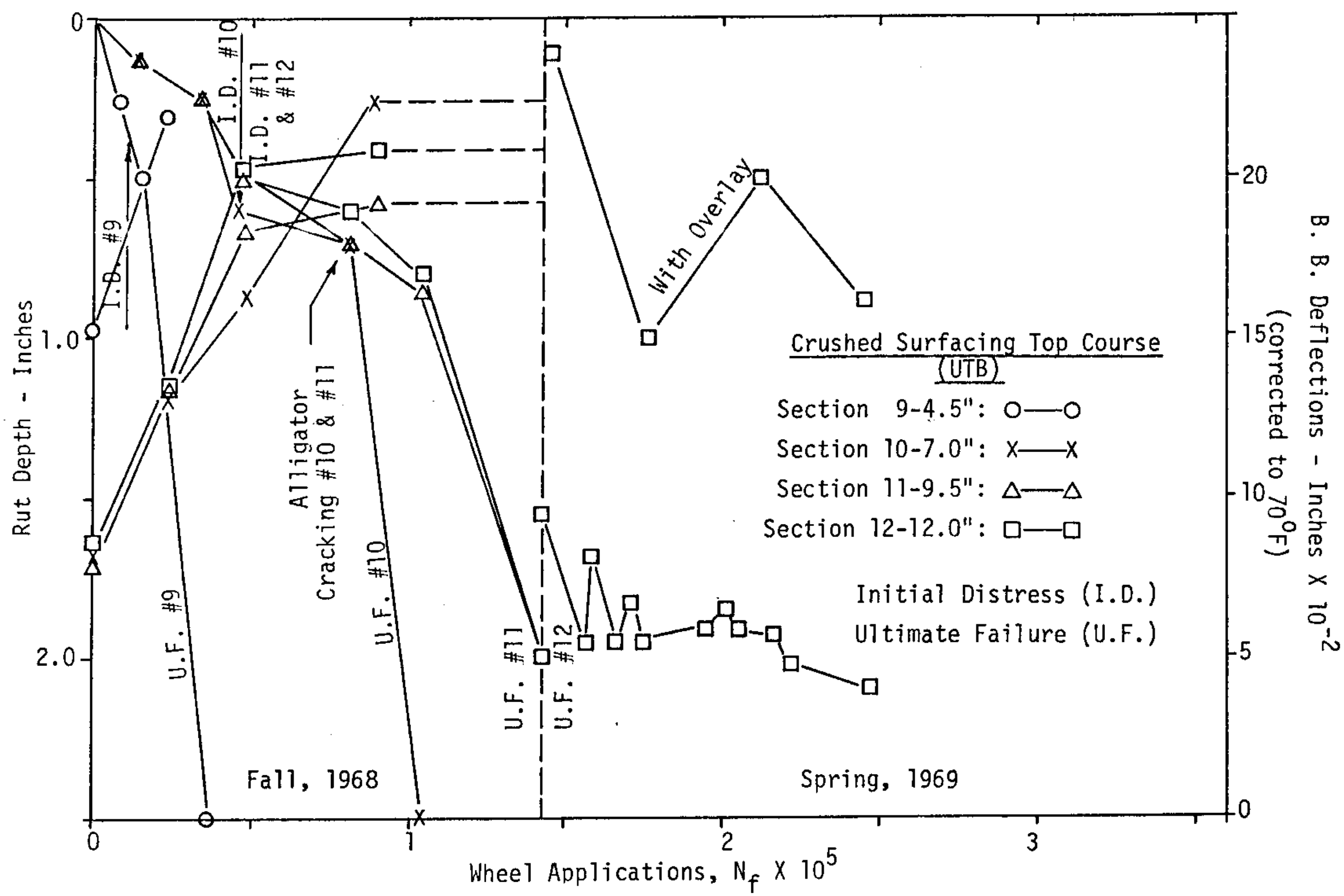


TABLE 15: PAVEMENT LIFE TO 0.25 INCH MAXIMUM RUTTING

Section	Ring 2			Ring 3			Ring 4		
	Base	Wheel Passes $Nx10^3$	Damage Ratio @ 0.25" Rut	Base	Wheel Passes $Nx10^3$	Damage Ratio @ 0.25" Rut	Base	Wheel Passes $Nx10^3$	Damage Ratio @ 0.25" Rut
1	UTB	150	19.95	ATB	550		SAB	104	
2	UTB	100	6.34	ATB	679		SAB	150	
3	UTB	100	0.26	ATB	736		SAB	160	
4	UTB	190	0.10	ATB	727		SAB	160	
5	ATB	210		UTB	379	31.7	ACB	47	
6	ATB	210		UTB	379	3.01 5.42 0.32	ACB	143.37	
7	ATB	210		UTB	550				
8	ATB	210		UTB	636		ACB	164.79	
9	ETB	99		ETB	500		UTB	10	6.63
10	ETB	99		ETB	736		UTB	37	1.24
11	ETB	210		ETB	740		UTB	37	0.19
12	ETB	210		ETB	750		UTB	37	0.03

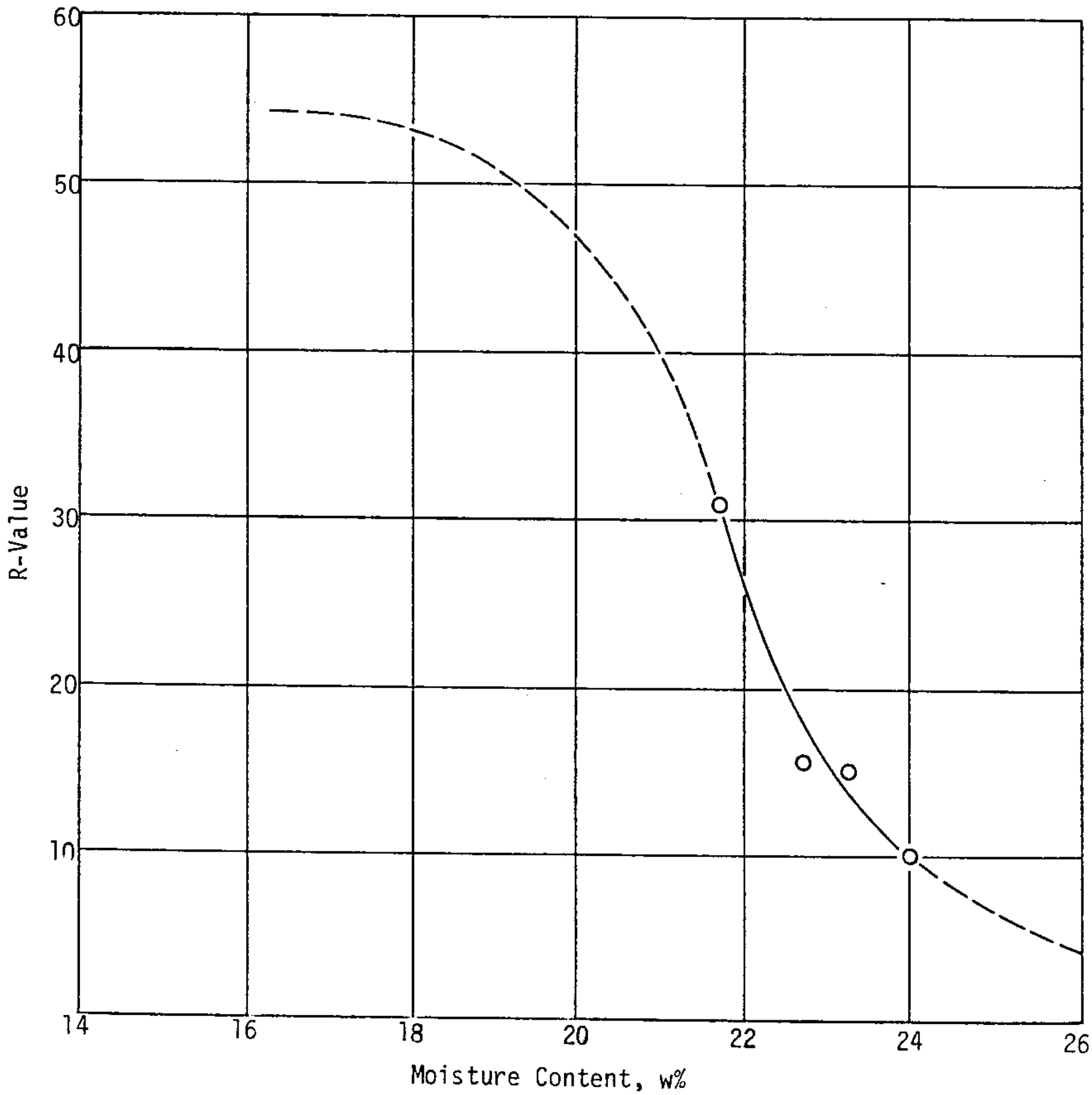


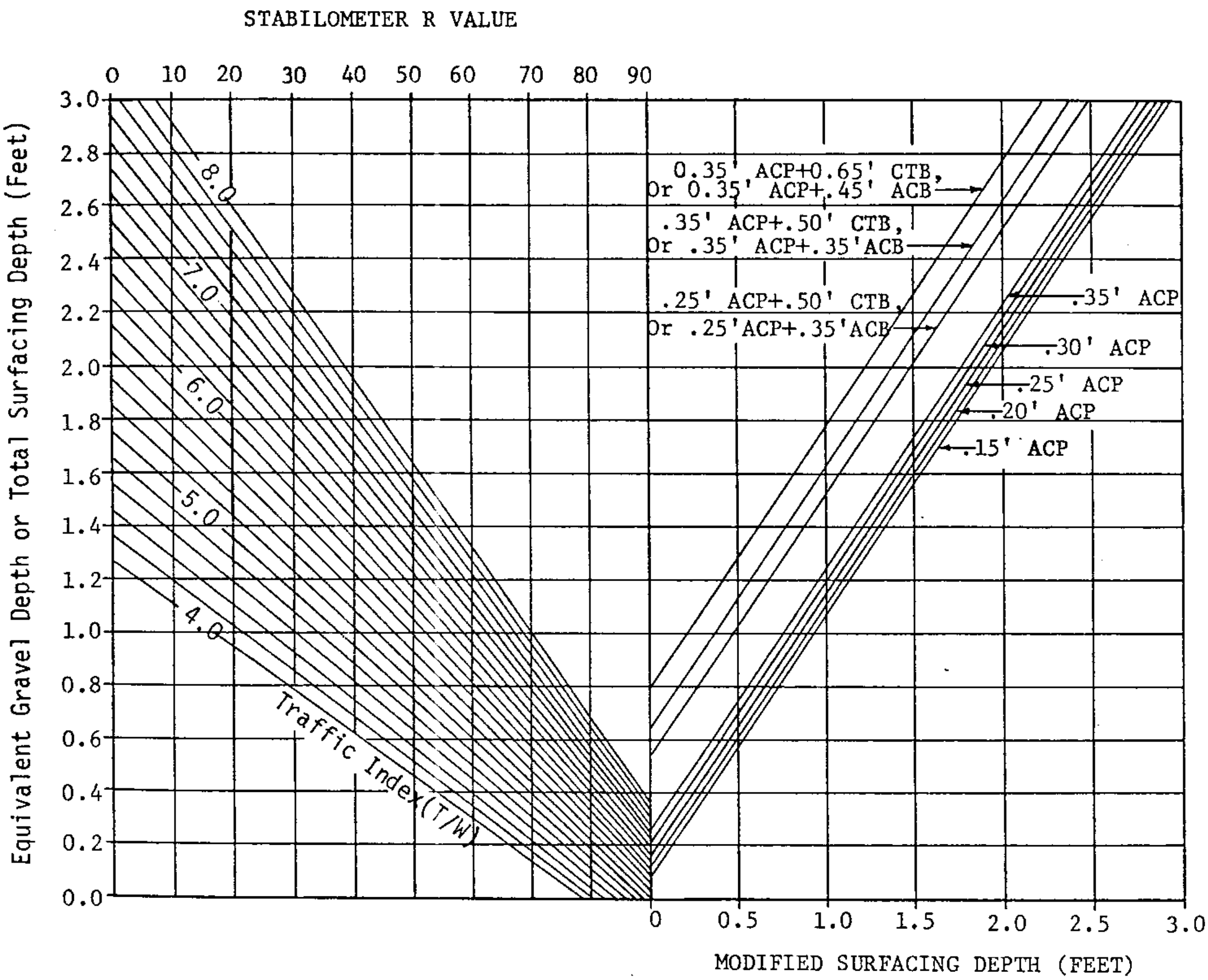
Figure 22: R-Value vs Moisture Content

results of this analysis were not encouraging. Not only were the numbers not equal to approximately 1.0--they ranged from 0.3 to 31.7.

An attempt was made to see if equivalency ratios could be selected for the different pavement structures used in the test track. Since the subgrade R-value was different for each section during its life, this must be taken into consideration. Therefore, it was decided to plot the average R-value vs. the number of wheel loads the section experienced until 0.25 inch ruts were recorded. Figure 24 shows this plot. The dotted lines in the figure were drawn from the Washington State Structural Design Chart for Flexible Pavements, which is reproduced as Figure 23 in this report.

Several observations can be made from Figure 24. First, the R-value does not seem to explain the differences in the life of the sections. The same approximate average R-value was experienced by most of the sections. However, in some cases there was a ten-fold difference in life span for the same structural section. They seem to be grouped by test ring number which indicates that conditions that existed during each test had the greatest influence on their life expectancy. The thickness and type of base also did not have as great an influence on the life of the pavement as indicated by the Washington State design chart. It is quite possible that the calculated average R-value is not a true representation of the actual R-value. Using the moisture content of the subgrade to calculate the R-value may not be adequate. The R-value vs. Moisture curve is also suspect since much extrapolation was necessary. However, it is not felt that the error in calculating the R-value is great enough to explain why the sections failed at relatively the same time. For instance, sections 4-10, 4-11, and 4-12 failed at about the same time. The calculated R-value for each section is about 50. However, if the R-value is to explain the same life span

Figure 23: Washington State Highway Structural Design Chart for Flexible Pavements



EXAMPLE:
Given an R value of 25 and a traffic index of 6.0, cover thickness requirements can be determined as follows:

An equivalent gravel depth of 1.65' (round to 1.7) at point A.

A modified surfacing depth of 1.05 feet at point B for a pavement of 0.35 ft. ACP + 0.50 ft. CTB, or .35 ft. ACP + .35 ft. ACB.

A modified surfacing depth of 1.43 ft (found to 1.45) for pavement of .35 ft. ACP.

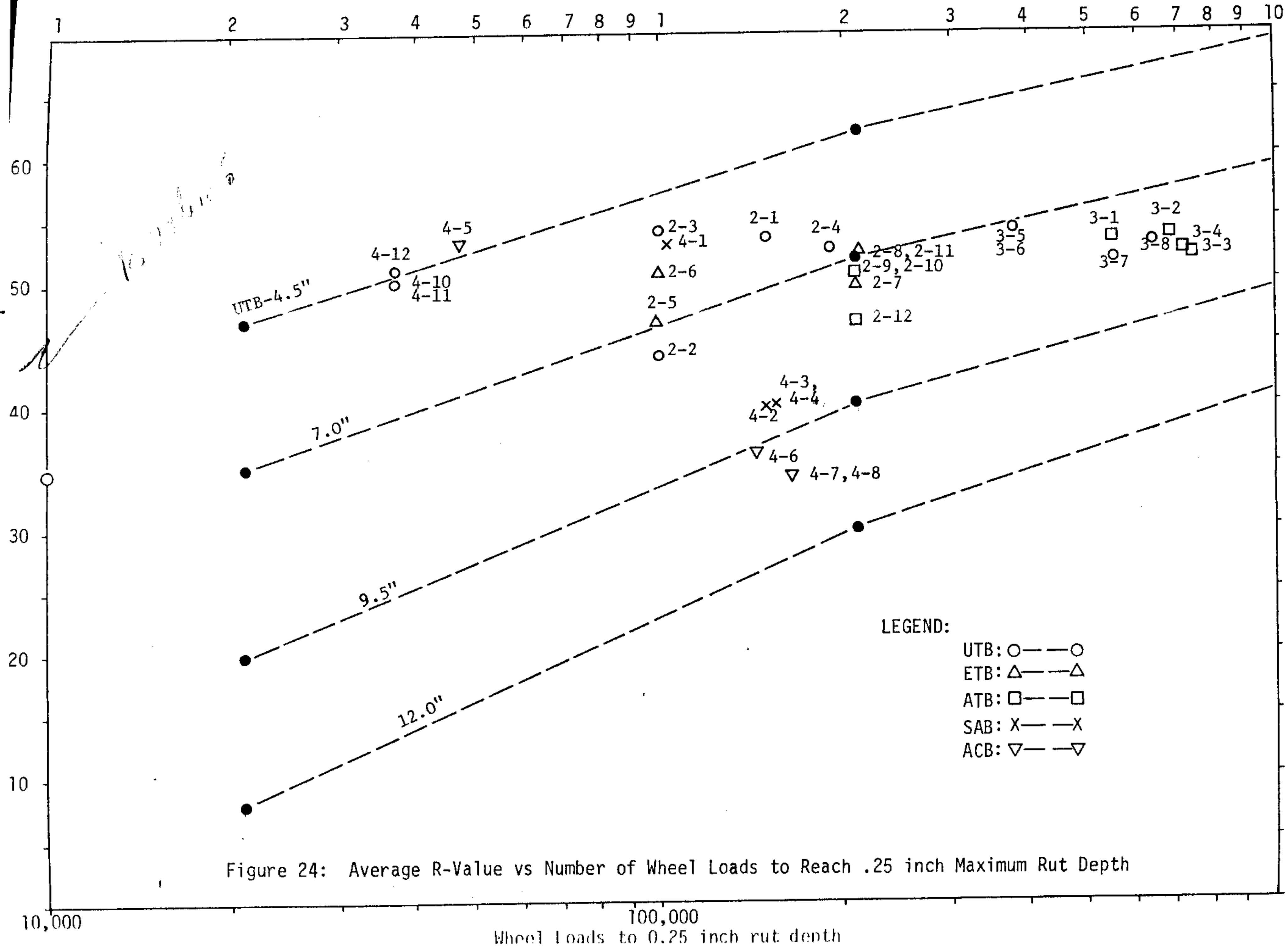


Figure 24: Average R-Value vs Number of Wheel Loads to Reach .25 inch Maximum Rut Depth

for each section, it should have been about 39 for section 4-10, 25 for section 4-11, and 18 for section 4-12. The error in calculating the average R-values is not this large.

The above discussion dealt with trying to explain rutting in terms of the strength of the subgrade. Obviously, many other factors also influence the rate and type of rutting that occurs. In 1971 Krukar [6] trenched across sections 3, 4, 8, and 12 of Ring 4 in order to determine the nature of the rutting that occurred in these sections. The study found that rutting occurred in the following manner:

a. densification of the Class "B" asphalt concrete wearing course and the base material in the wheel path;

b. lateral movement of the material under the wheel path to the untravelled areas; and

c. deformation of the subgrade.

Densification of the asphalt layers appeared to be a major contributor to rutting, as illustrated in Table 16. In fact, the density of the asphalt in the wheel path increased an average of 10 lb/cu ft. Reduction in pavement thickness is not completely explained by densification. Much of this reduction was probably due to lateral shoving of the mix from under the wheel path to the sides. The last column of Table 16 lists the approximate amount of reduction in pavement thickness due to shoving. The amount shown is only an approximation since the initial thickness is an average for the entire Ring. Local variations in thickness may explain much of the scatter in the data. The great variation in asphalt cement content, as noted earlier, may also explain some of the scatter observed in the data. In section 3, with the 6.0 inches of S.A.B., some deformation of the subgrade took place. This is further amplified in reference 4.

TABLE 16: CHANGES IN THICKNESS AND DENSITY IN WHEEL PATH, Ring 4

Section	Thickness (inches)		Density (pfc)		Change in Thickness (in.)		
	Initial	Trench[1]	Initial	Trench	Total Observed	Due to Densification	Due to Shoving
	(a) Wearing Course						
3	3.16	2.50	146.3	157.5	-0.66	-0.22	-0.44
4	3.16	2.81	146.3	156.3	-0.35	-0.20	-0.15
8	3.16	3.14	146.3	155.4	-0.02	-0.19	+0.17
12	3.16	2.40	146.3	156.4	-0.76	-0.20	-0.56
	(b) Base						
3	5.94	5.38	118.1	122.1	-0.56	-0.19	-0.37
4	8.19	7.95	115.7	119.8	-0.24	-0.21	-0.03
8	5.16	4.42	150.4	149.2	-0.74	+0.04	-0.70
12	12.00	11.70	134.2	136.3	-0.30	-0.19	-0.11

[1] Thickness determined at maximum net depth.

Although Hveem developed the Stabilometer method to design pavements against permanent deformation, the method has been extended to account for fatigue characteristics also.

In order to check the validity of extending Hveem's basic assumptions to include fatigue life, the number of wheel loads to the initial distress as recorded in Tables 9, 10, and 11 was compared to the average R-value. The initial distress is defined as that time when the first cracks appear at the surface of the pavement. The number of wheel loads to initial distress was not significantly different from the number of wheel loads to a rut depth of 0.25 inches. Essentially, the same relationships as those shown in Figure 24 are evident. Therefore, conclusive evidence on the validity of extending the Hveem design procedures to include fatigue life is not possible from the data available to the investigators.

ELASTIC THEORY

Hveem [13] recognized the limitation of the R-value design method to predict fatigue failure in the pavement. He stated other methods should be used to explain this phenomenon. During the past several years, many researchers have turned to the elastic theory to see if fatigue cracking can be predicted. Results have been very favorable. Kingham and Kallas [7] conducted an extensive analysis of fatigue failure in Ring 4. The cumulative damage hypothesis of Miner was used with very good success. Tables 17 and 18 were reproduced from their report to show the accuracy of their analysis. Major conclusions of this analysis are as follows:

1. Laboratory-fabricated test specimens for fatigue tests did provide the same results as those cored from untraveled portions of the test pavements. This finding suggests that lab fabrication can produce a specimen representative of the field.

Table 17

PREDICTED AND OBSERVED PERFORMANCE, RING 4

Base Type	Section	Surface and Base Thickness in.	Failure Criteria--Repetitions to Failure				Actual[1]
			Lab. Controlled Stress	Lab. Controlled Strain	Field, AASHO Moduli	Field, WSU Moduli	
Sand Asphalt	1	3-2	132,100	(162,784+)	3,400	151,700	144,660
	2	3-4	(190,801+)	(190,801+)	9,100	(190,801+)	159,789
	3	3-6	(220,189+)	(220,189+)	54,000	(220,189+)	175,620
Asphalt Concrete	5	3.1-0	153,300	157,100	3,400	14,800	47,391
	6	3.1-2	164,800	(158,137+)	14,800	103,000	148,887
	7	3.3-3.5	174,900	(170,710+)	82,500	165,800	161,262
Crushed	9	3-4.5	147,000	152,700	390	3,400	12,000
	10	3-7.0	150,300	156,700	390	3,400	47,391
	11	3-9.5	153,700	158,000	390	3,400	48,000
	12	3-12	156,700	160,100	390	3,400	49,104

(+) Further analysis periods are required for the failure prediction since cycle ratios were less than .85 at the end of computation.

1 After Krukar and Cook, Reference [4].

(After Kingham and Kallas)

Table 18

PREDICTED PERFORMANCE COMPARISON USING LOWER SUBGRADE MODULUS LABORATORY, CONTROLLED STRESS FAILURE CRITERIA, RING 4

Base Type	Section	Surface and Base Thickness, in.	High E_s [1]		Low E_s [2]		Observed[3]
			Damage Ratio	Repetitions	Damage Ratio	Repetitions	
Sand Asphalt	1	3-2	1.0	132,100	1.0	25,200	144,660
	2	3-4	.85	190,800	1.0	90,700	159,789
	3	3-6	.34	220,200	.52	153,700	175,620
Asphalt Concrete	5	3.1-0	1.0	153,000	1.0	67,800	47,391
	6	3.1-2	1.0	164,800	1.0	158,000	148,887
	7	3.3-3.5	1.0	174,900	1.0	166,800	161,262
Untreated	9	3-4.5	1.0	147,000	1.0	18,700	12,000
	10	3-7.0	1.0	150,300	1.0	22,200	47,391
	11	3-9.5	1.0	153,700	1.0	25,200	48,000
	12	3-12.0	1.0	156,700	1.0	25,200	49,104

1. High Subgrade Modulus E_s
   - Asphalt Bases 15,700 psi
   - Crushed Stone Base 7,700 psi
2. Low Subgrade Modulus E_s
   - Asphalt Bases 6,800 psi
   - Crushed Stone Base 3,800 psi
3. Observed number of load repetitions to first cracking, Krukar and Cook[5]

2. Predictions from the stress controlled laboratory tests were very close to those observed for 3 of 6 full-depth asphalt test sections.
3. Stress-controlled laboratory fatigue tests predicted the life of full-depth asphalt pavements better than strain-controlled tests. Both stress modes tended to over-predict full-depth asphalt pavement life.
4. Field failure criteria developed from AASHO Road Test data gave slightly closer predictions than laboratory results. Promise exists for the application of the criteria to asphalt mixtures that have different dynamic modulus - temperature relationships than the AASHO Road Test bituminous base.
5. Neither the laboratory fatigue data nor the field failure criteria predicted satisfactorily the lives of the crushed stone-base test sections.

The difficulties in predicting the lines of the crushed stone base test sections could have been alleviated if, in addition to the maximum deflection, the shape of the deflection bowl had been defined. Since the radial strain on the bottom of the asphalt layer is used to determine fatigue life, knowing the shape of the deflection bowl is very important. Other investigators have found [14] that the bowl shape is most influenced by the modulus of the upper layers in the pavement structure. One modulus was used to represent the entire subgrade. Because the moisture content of the subgrade was greatest in the upper twelve inches and decreased with depth, several modulii should have been used.

The analysis of Kingham and Kallas primarily dealt with the test sections of Ring 4. Extension of the procedures used to Rings 2 and 3 was not possible for the following reason as explained by Kingham and Kallas:

Further analysis for test sections 2 and 3 would have required consideration of load repetitions applied when pavement temperatures at the bottom of the asphalt layers were in excess of 80°F. Since laboratory fatigue curves for the asphalt bases were obtained at two temperatures, the highest being 60°F, the validity of any analyses considering high temperatures was questionable. The statistical models used for the asphalt bases assumed a linear temperature effect which was satisfactory for interpolation purposes but suspect for extrapolations of more than 15° to 20°F. [7]

EVALUATION OF OTHER DESIGN PARAMETERS

There are other parameters which should be considered in flexible pavement design. These include deflections, strains, stresses, and rut depths. These are examined here and compared with other studies.

Some of the most important parameters studied at the AASHO Road Test and elsewhere have been deflections and deflection basins. At the WSU Test Track, both static rebound Benkelman beams and dynamic deflections have been studied. The latter has also been computed by using a computer solution developed by Chevron Research Corporation [15] for a semi-infinite layered elastic system. Comparison of computed dynamic deflections with dynamic deflections measured with LVDT gauges has been very good and has been reported on by Terrel [16], and Krukar and Cook [17].

It is known that deflections depend upon the pavement structure which is very temperature susceptible. Deflections will change with the time of the day as reported by Coffman et al. [18] and with the seasons as reported by the Canadian Good Roads Association Design Manual [19]. The study by

Coffman et al. shows that equivalencies based on deflections can vary quite drastically. Deflection studies at the WSU Test Track show definite differences [4]. This points out the continually changing modulus of the asphalt pavement layers with temperature. Figures 25, 26, 27, and 28 from Test Ring #2 show the variations of dynamic deflection with temperature. Deflections will vary with season due to changing environmental conditions.

Certain "critical" design values of deflections have been noted. The results of the AASHO Road Test, as interpreted by the CGRA Observer Committee [20], showed that during a 2-year test period, when this rebound value exceeded 0.05 inches, surface cracking developed followed by pavement failure. Distress did not occur when the rebound value remained below 0.05 inches. The results from the test track as shown in Tables 9, 10, and 11, pages 26-28, and in Figures 19, 20, 21, pages 45-47, confirm this. The pavements with high rebound deflections failed rapidly. Dynamic deflection values also indicate this although normally these are lower than the static rebound deflections. But the trends are there, as shown in Figures 29 and 30, pages 69 and 70, as well as the relationship between the static rebound deflections and dynamic deflections. Figures 25, 26, 27, and 28 also confirm that high dynamic deflections will result in failures sooner than anticipated. It should be remembered that test rings #2-4 were designed so that rapid failures occurred, and hence the high deflection values reflect the research objectives. It is unfortunate that deflection was not measured in Ring #1 which had the thick pavement structures. This probably would have confirmed other studies.

Since deflections are a good measure of the pavement structure and also subgrade conditions, the Asphalt Institute uses a rebound deflection method for the design of overlays [21]. From the above discussion, the AASHO Road Test, and CGRA studies, a good "critical" rebound deflection value to use

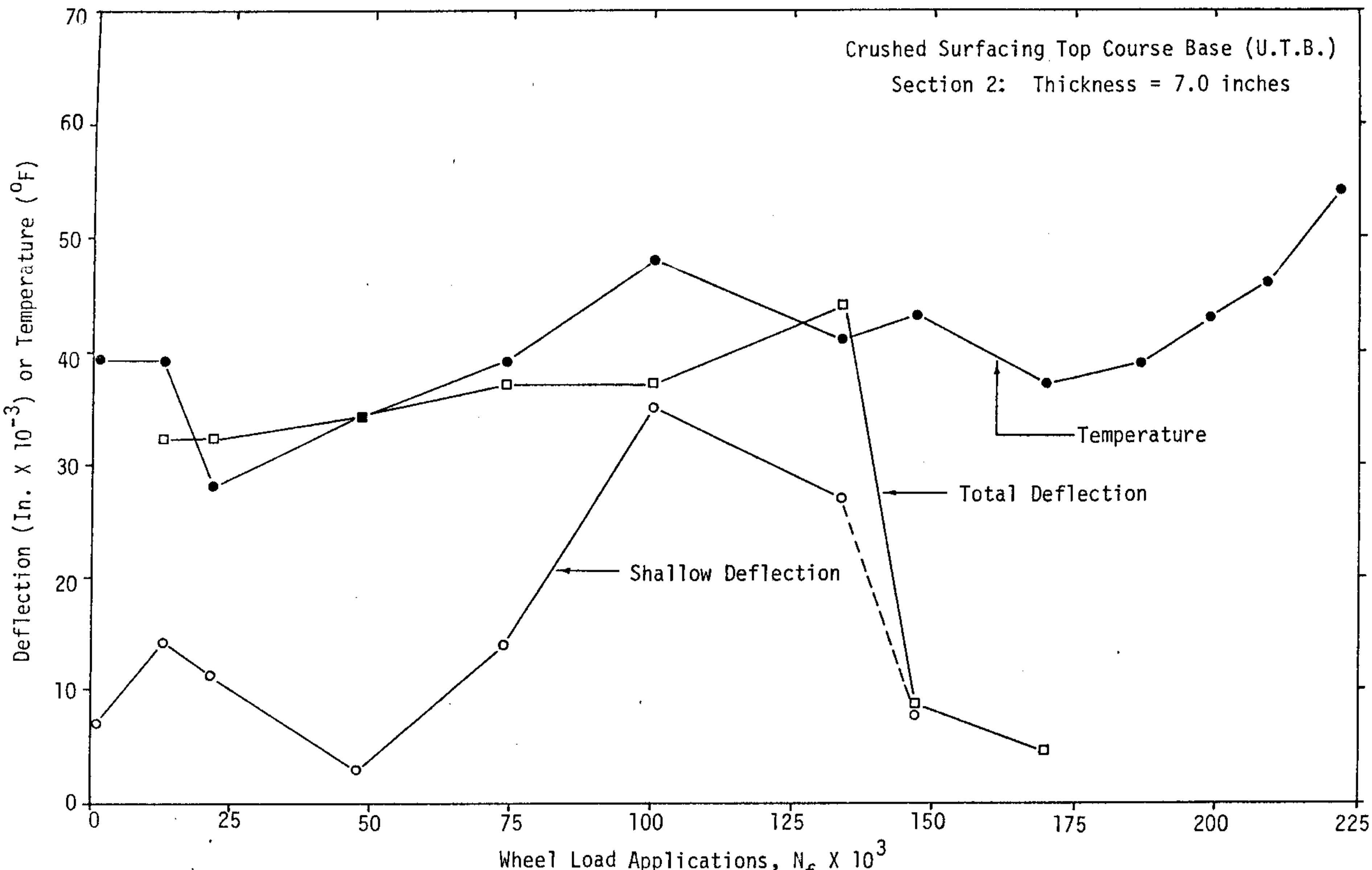


Figure 25: Deflection and Temperature vs. Wheel Load Applications, Ring 2 - Untreated Base

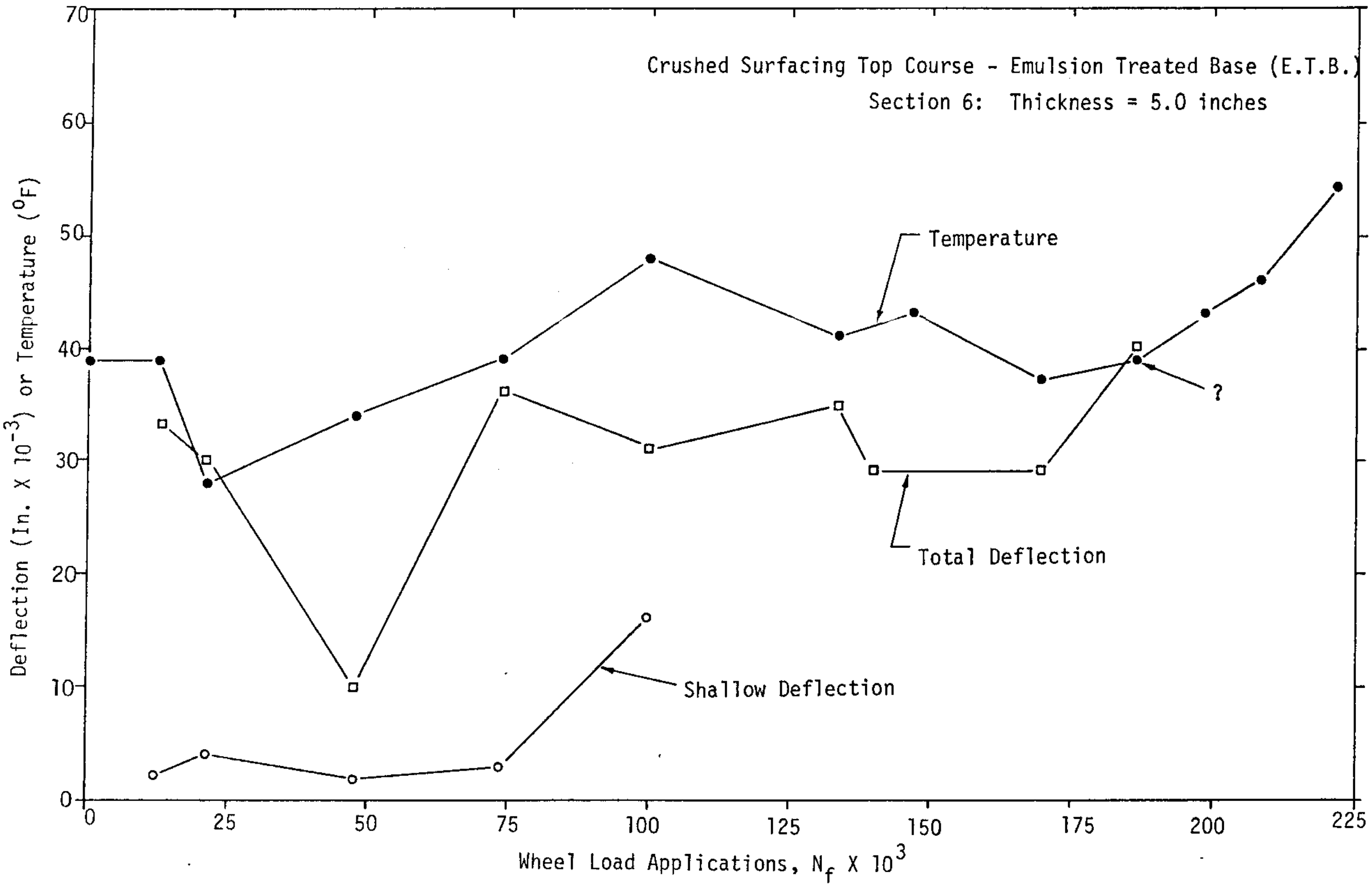


Figure 26: Deflection and Temperature vs. Wheel Load Applications, Ring 2 - Emulsion Treated Base

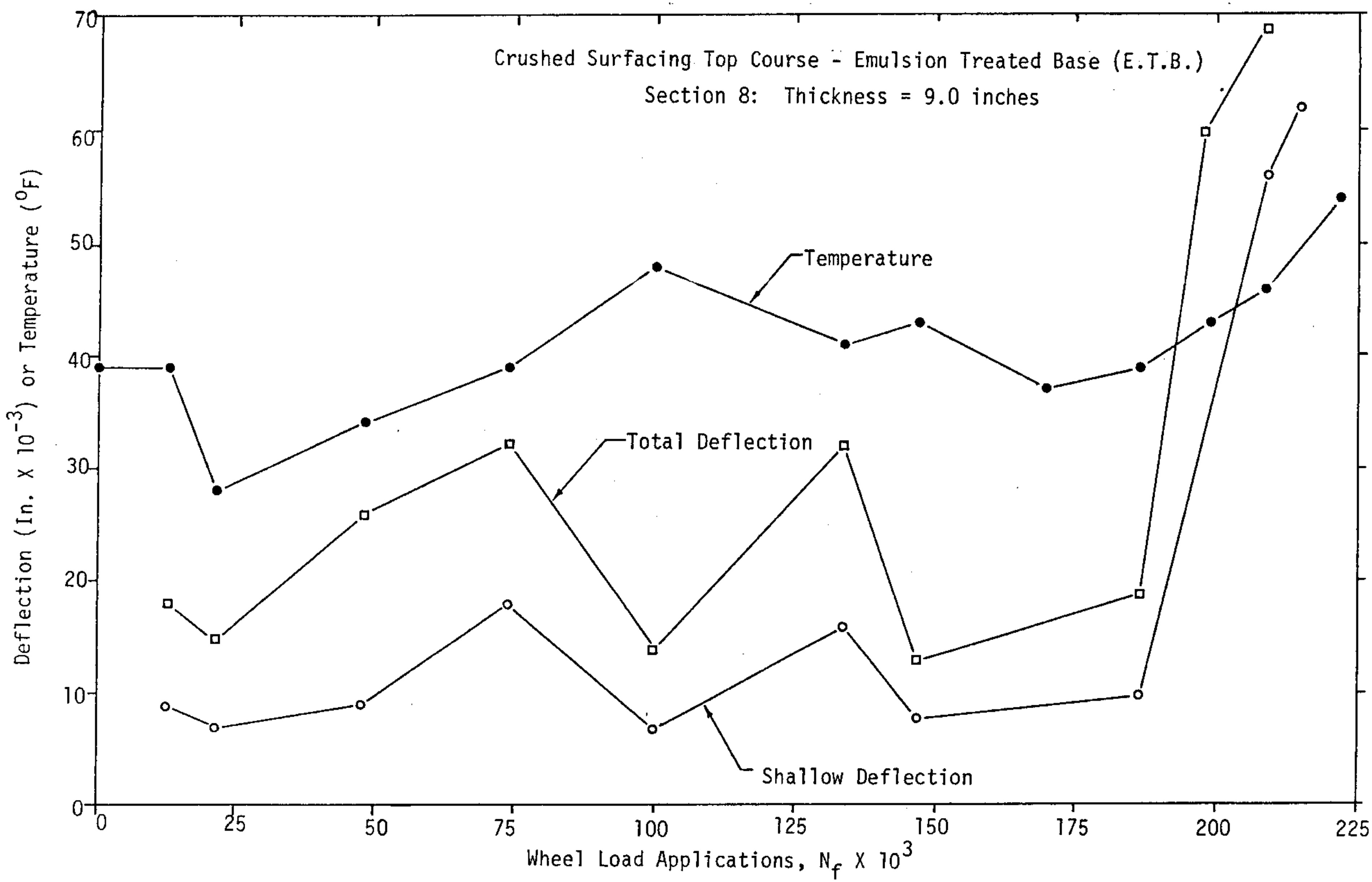


Figure 27: Deflection and Temperature vs. Wheel Load Applications, Ring 2 - Emulsion Treated Base

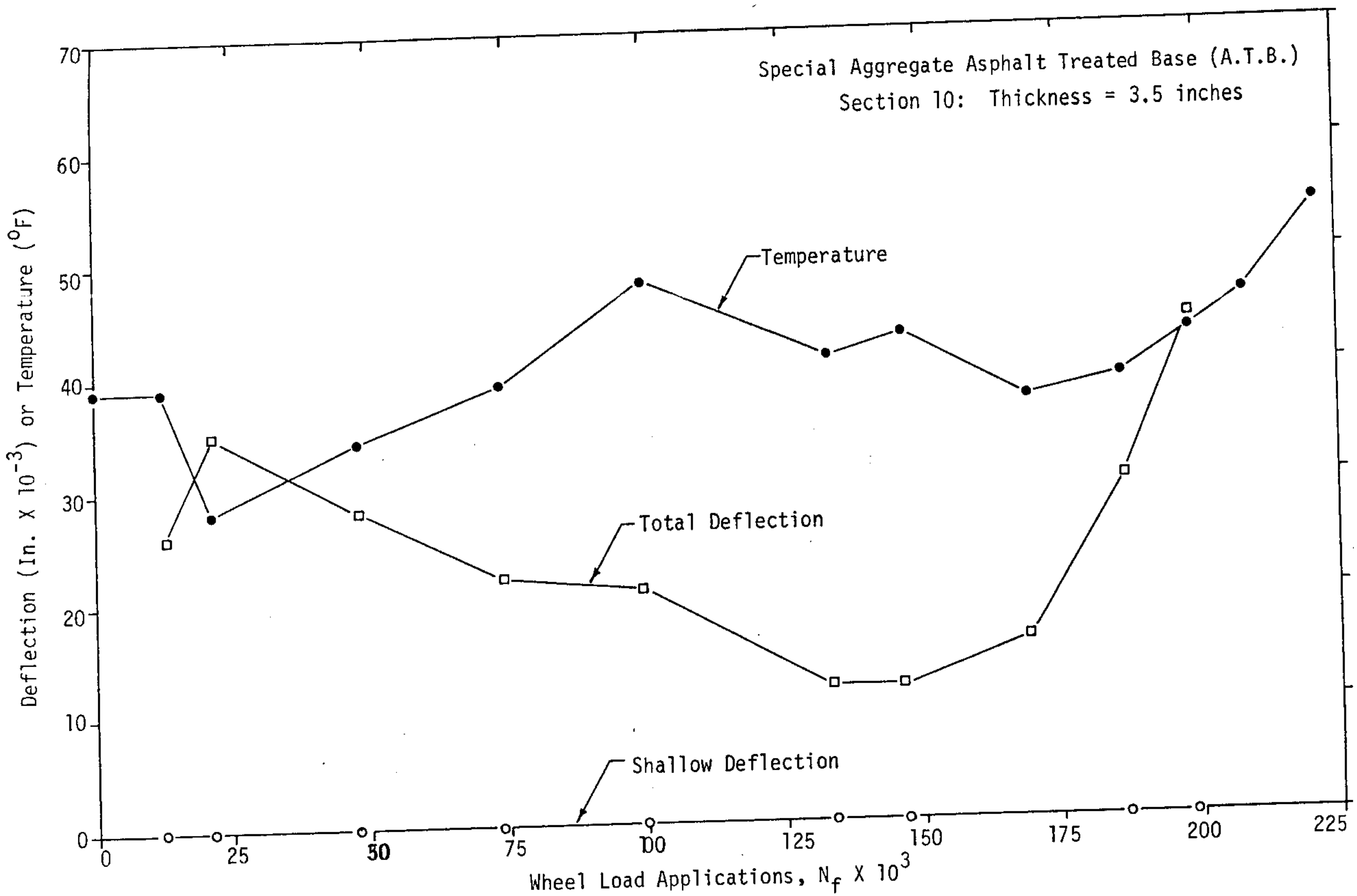


Figure 28: Deflection and Temperature vs. Wheel Load Applications, Ring 2 - Asphalt Treated Base

would be 0.050 inches, and roads be designed for 0.030 to 0.050 inches maximum spring Benkelman rebound value [19].

The problem of developing a series of "critical" strain levels for the different base materials is very difficult. It is difficult in the sense that there are no "critical" values in the true meaning of the word "critical." At a certain strain level for a certain pavement, the pavement will have an expected life before it will start to fail. If the strain level is increased, by increasing the traffic and hence the loading, the pavement life is correspondingly reduced. In this sense, there are no "critical" strain values. Thus, increasing the thickness of the pavement will increase pavement life by reducing the strain level and vice-versa. Another problem is that strain values will vary with temperature as the modulus of the asphalt pavements change. The designer using this design method will have to assume an average temperature level for the pavement structure and use the corresponding modulii values as determined from laboratory tests and data. From this he can then calculate the strain levels and then predict the life of the pavement. This means that extensive laboratory fatigue and stress-strain tests should be run on the type of asphalt pavement with corresponding materials and the subgrade material. Modulus curves at different temperatures can be developed. A thickness design method can then be developed. One also has to predict the expected traffic. This, too, is critical in that if one predicts a very low traffic and large traffic develops, the pavement will develop earlier failures than anticipated. Values used for design purposes are shown in references 16 and 17 and are:

Strain in bottom of: ATB, ACB	150×10^{-6} in/in.
Surface	300×10^{-6} in/in.

These are higher than might be normally used for design purposes. Depending upon the thickness of the asphalt pavements, the engineer has to decide whether the pavement is in a controlled-stress or a controlled-strain loading mode. Monismith et al. [22] suggests that the latter is suitable for defining response of mixtures in thin pavement (2 inches or less of asphalt concrete) while the former appears to be suitable for comparatively thick pavements. The results from the AASHO Road Test and other recent findings substantiate earlier findings that the load spreading abilities of flexible pavements with conventional, untreated bases are very limited. At normal temperatures and under slowly moving loads, the measured vertical stresses generally follow the pattern predicted by the Boussinesg theory for a homogeneous solid. Directly under the load, the vertical stresses are considerably higher than those predicted by the conventional layered solid (Burmister) theory. They are highest when the subgrade, as well as the pavement structure, is practically saturated with moisture; they are lowest during frost periods. The stresses are also affected by the vehicle speed, being lower under post-morning loads. This was shown in test track ring 2 [2].

It can be said that the pavement acts as a very complex layered solid which, because of lack of tensile strength of some layers, exhibits only very limited slab action. However, this action is considerably increased when all pavement layers become frozen and acquire greater strengths. These greater tensile strengths are easily achieved by a treatment with cement or bitumen, hence the case for treated bases.

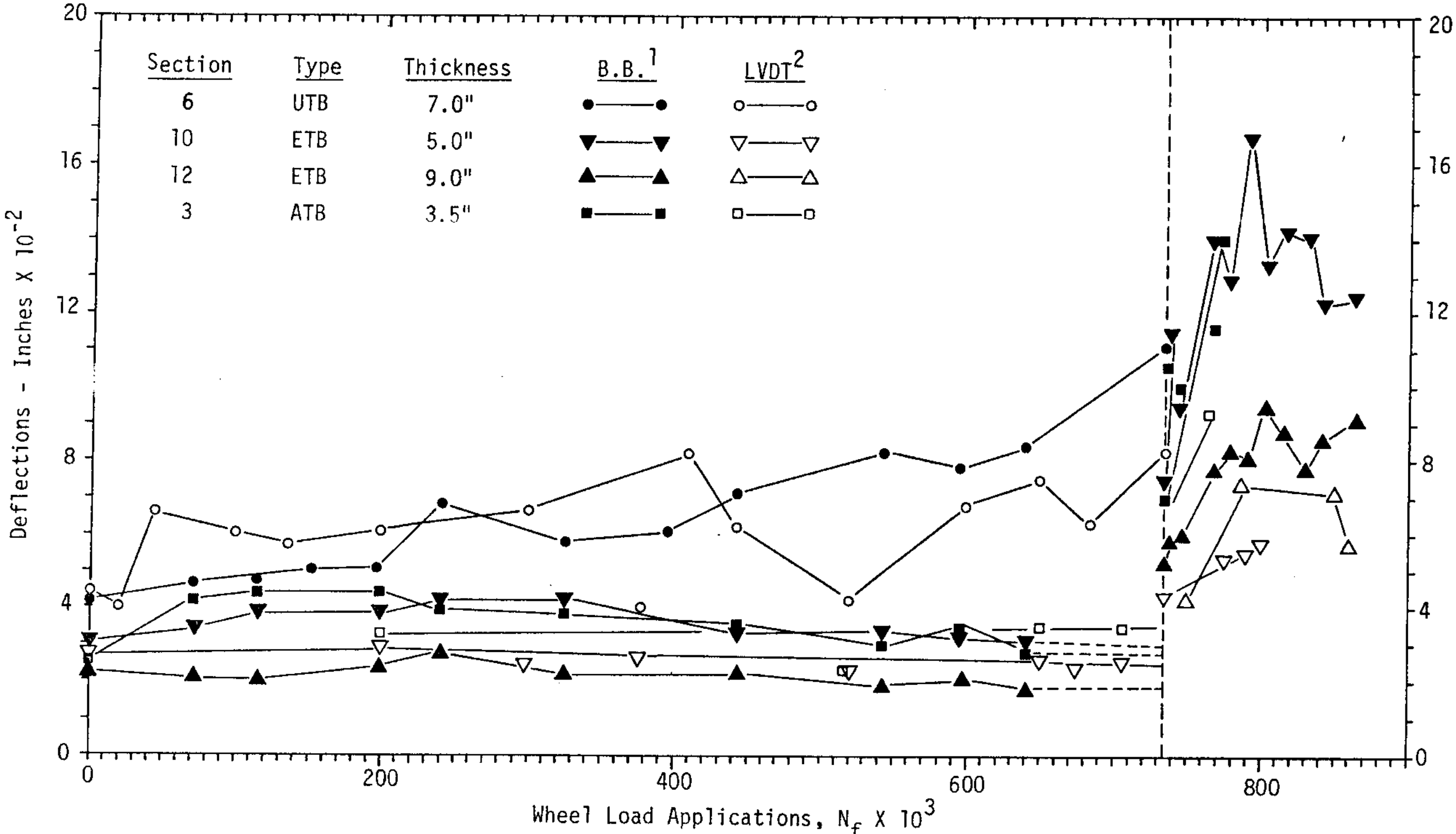
The above discussion points out that vertical stresses can vary with environmental factors. Pressure cells in Ring #2 and also computed values point out these differences. Vertical stresses at the top of the subgrade were so much greater in the spring than in the fall as shown in Figures 31, 32, 33, and 34, pages 71-74. Thickness and temperatures also affected these values. The latter affected the elasticity modulii of the asphalt pavements; the higher the temperature, the lower the modulus. This lowered the stiffness of the asphalt concrete. Similar findings are shown in Figures 34, 35, and 36 for Ring #3 and 37, 38, and 39 for Ring #4.

Vertical stresses at the top of the subgrade, measured and computed, show the same trend. They were generally higher under the untreated bases and lowest under the asphalt treated and asphalt concrete bases. All these figures, 31 to 37, show that pavements which had vertical stress at the top of the subgrade of 10 psi or more had earlier failures. This was especially true for the thinner sections. In the spring, the combination of high vertical stresses with saturated subgrades and high pavement temperatures resulted in sudden failures [2, 3, and 4]. From the above discussion it would seem advisable for the highway engineers to keep vertical stress at the top of the subgrade below 10 psi, and preferably below 5 psi.

This has an important bearing on factors influencing rut depth. As mentioned earlier, excavation of the sections in Ring 4 showed that rutting under WSU conditions was due to a combination of 3 factors: a) densification of the wearing course, b) lateral shoving due to the wheels, and c) deformation of the subgrade. Table 16 shows this for several of the Ring 4 sections.

The fact is that ruts will occur as a result of load repetitions, and they will stabilize [23]. This may be due to surface densification and/or due to the fact that the soils and granular materials in gravel have been subjected to stresses which are well below the ultimate strength of the material. The results from the AASHO Road Test illustrate that even "structurally adequate" pavements will develop appreciable rutting under a large number of load repetitions and point out the need for consideration,

Figure 29: Deflection Trends with Wheel Applications, Ring #3

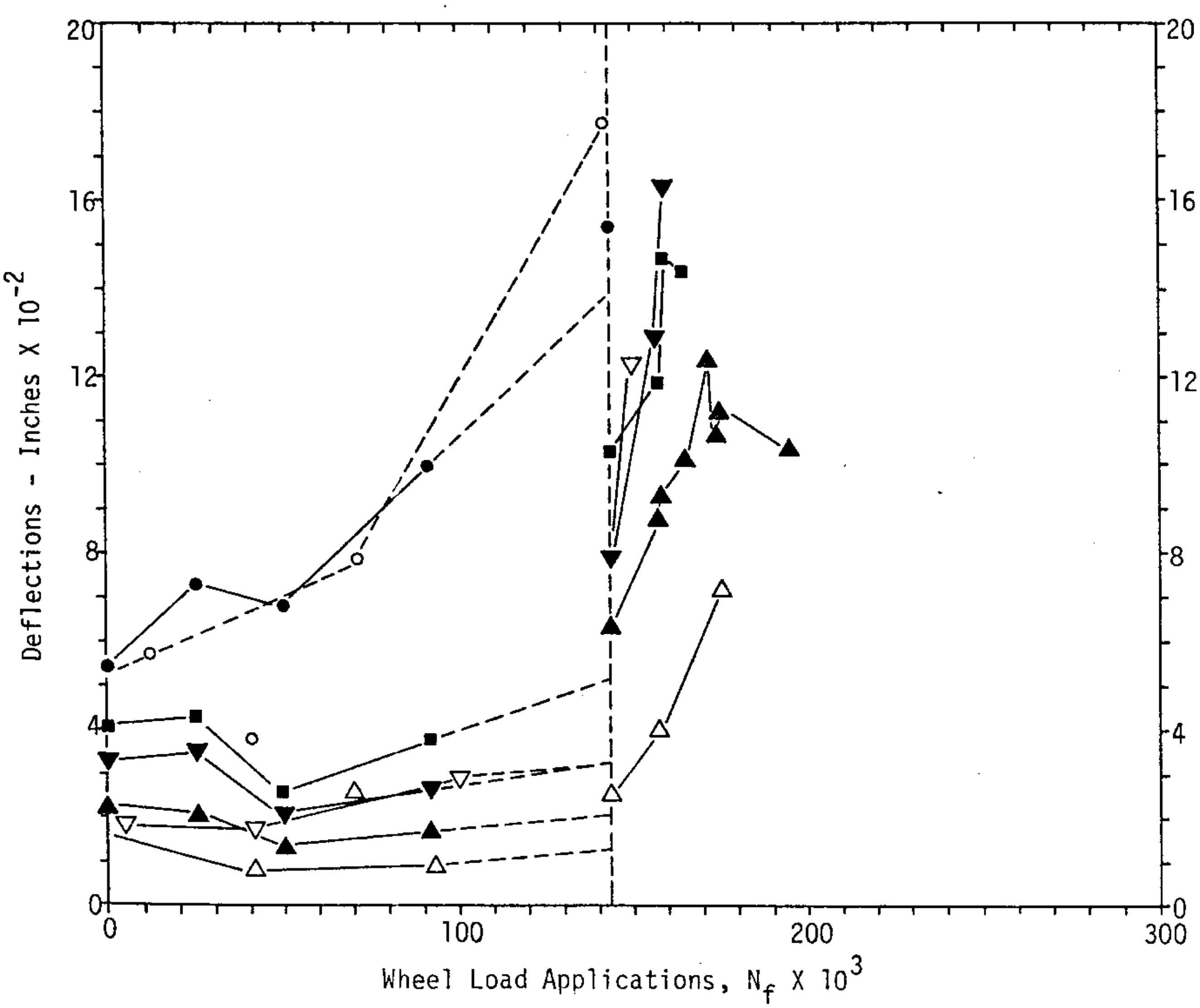


[1] Benkelman Beam deflections uncorrected

[2] LVDT Total Deflection

Figure 30: Deflection Trends with Wheel Applications, Ring #4

Section	Type	Thickness	B.B.[1]	LVDT[2]
10	UTB	7.0"	●——●	○——○
2	SAB	2.0"	▼——▼	▽——▽
4	SAB	8.0"	▲——▲	△——△
7	ACB	3.5"	■——■	□——□



[1] Benkelman Beam deflections uncorrected

[2] LVDT Total Deflection

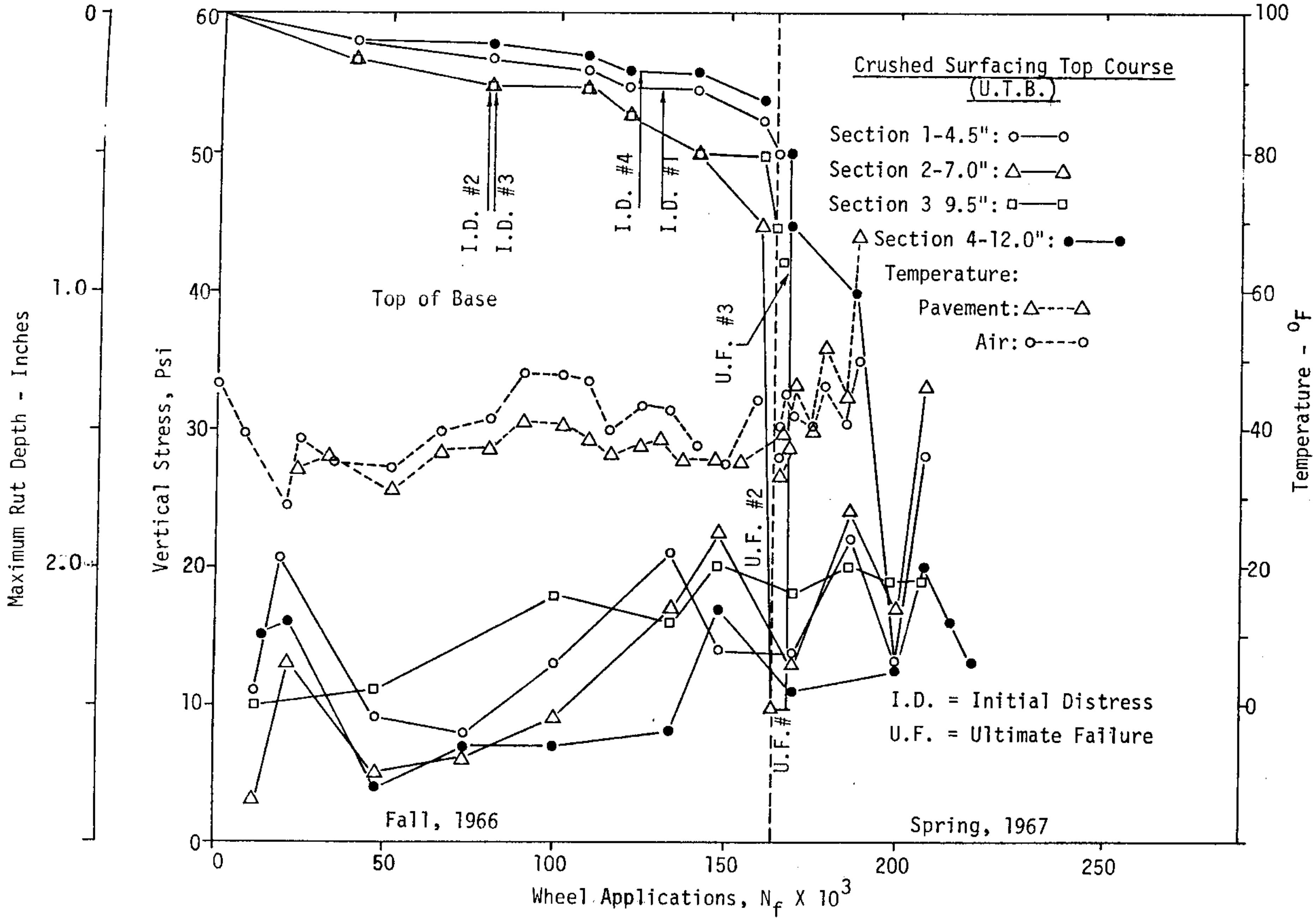


Figure 31: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature With Wheel Applications for Pavements with Untreated Base (U.T.B.), Ring #2

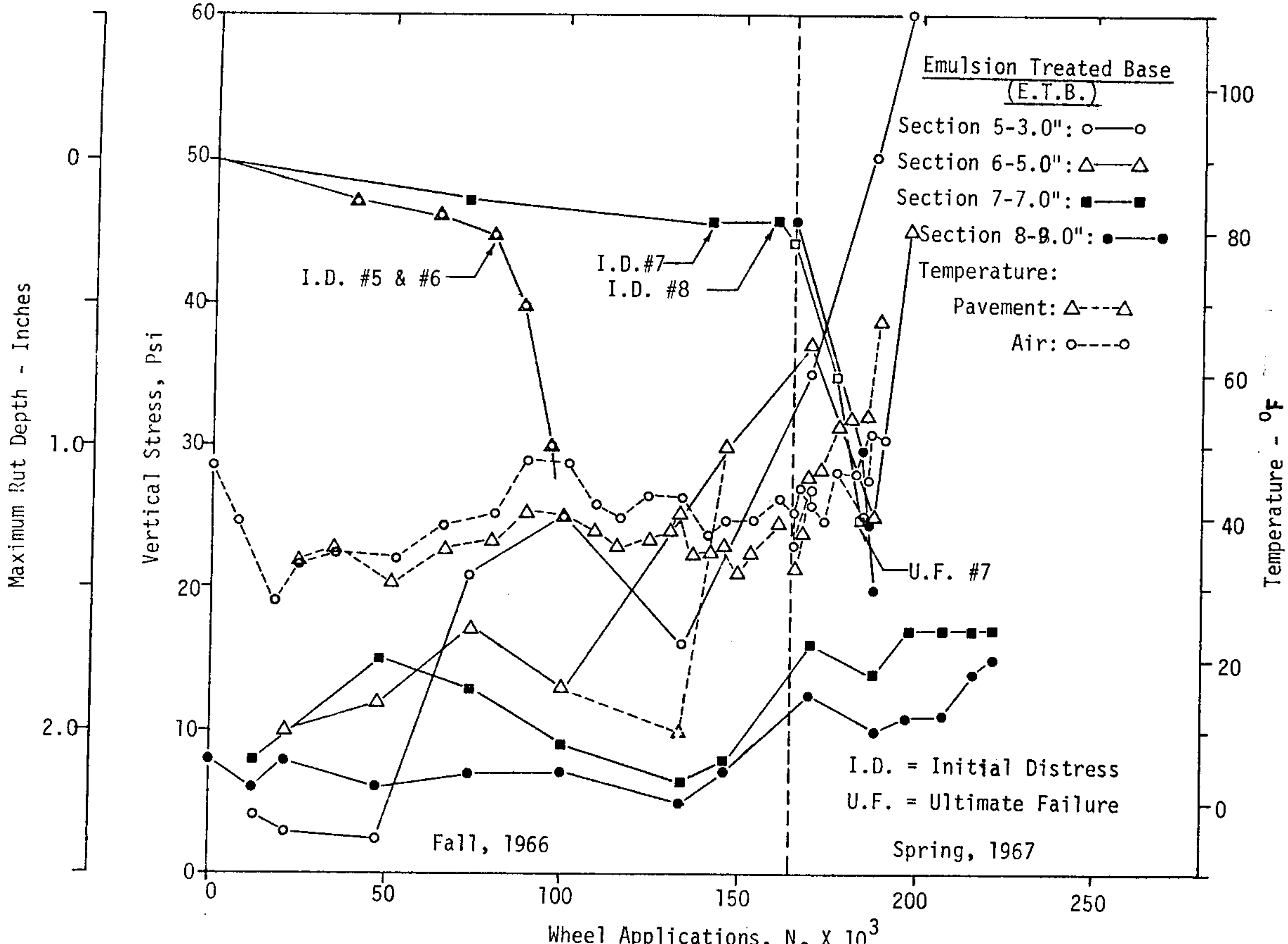


Figure 32: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature With Wheel Applications for Pavements with Emulsion Treated Base (E.T.B.), Ring #2

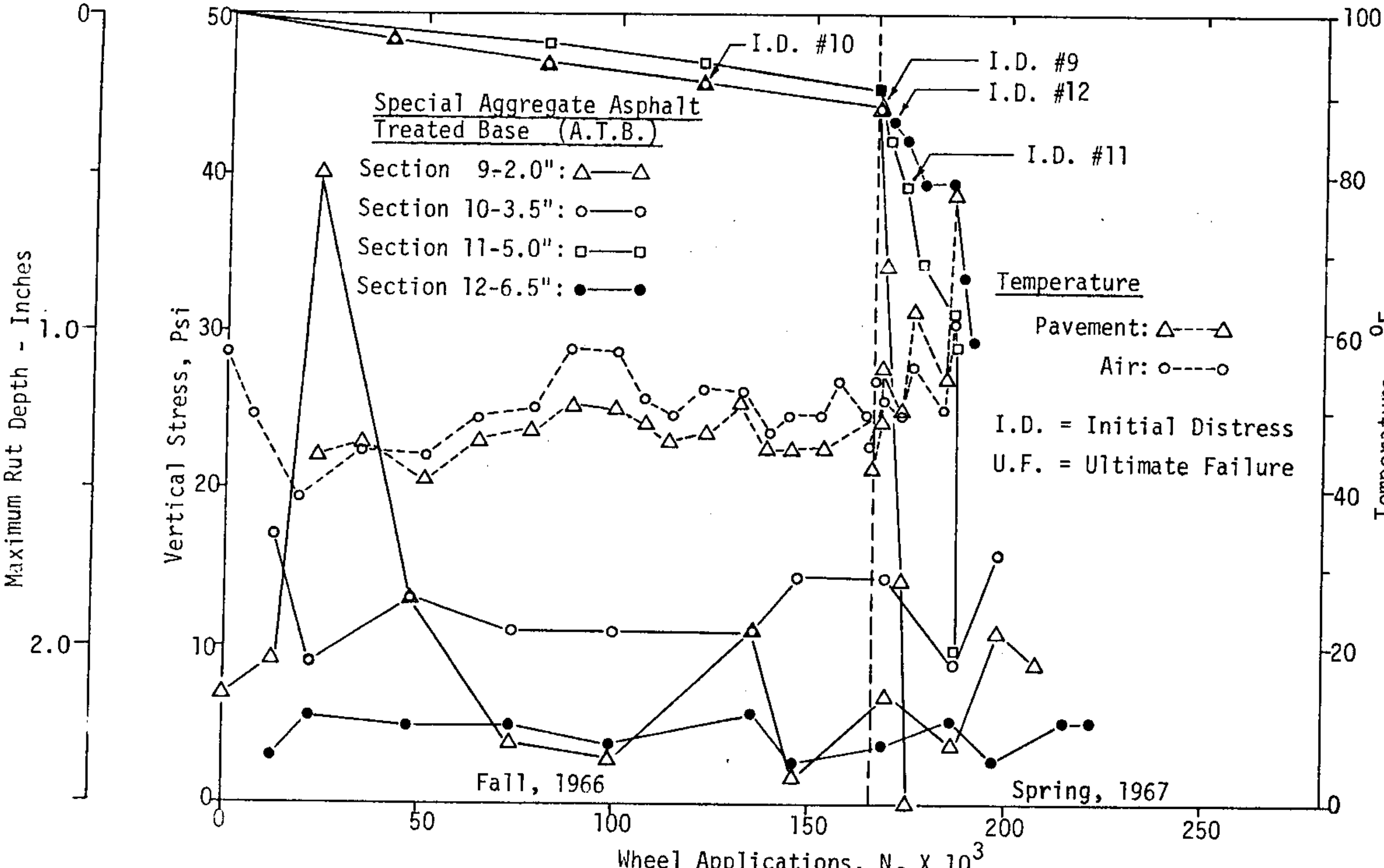


Figure 33: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature With Wheel Applications for Pavements with Special Aggregate Asphalt Treated Base (A.T.B.), Ring #2

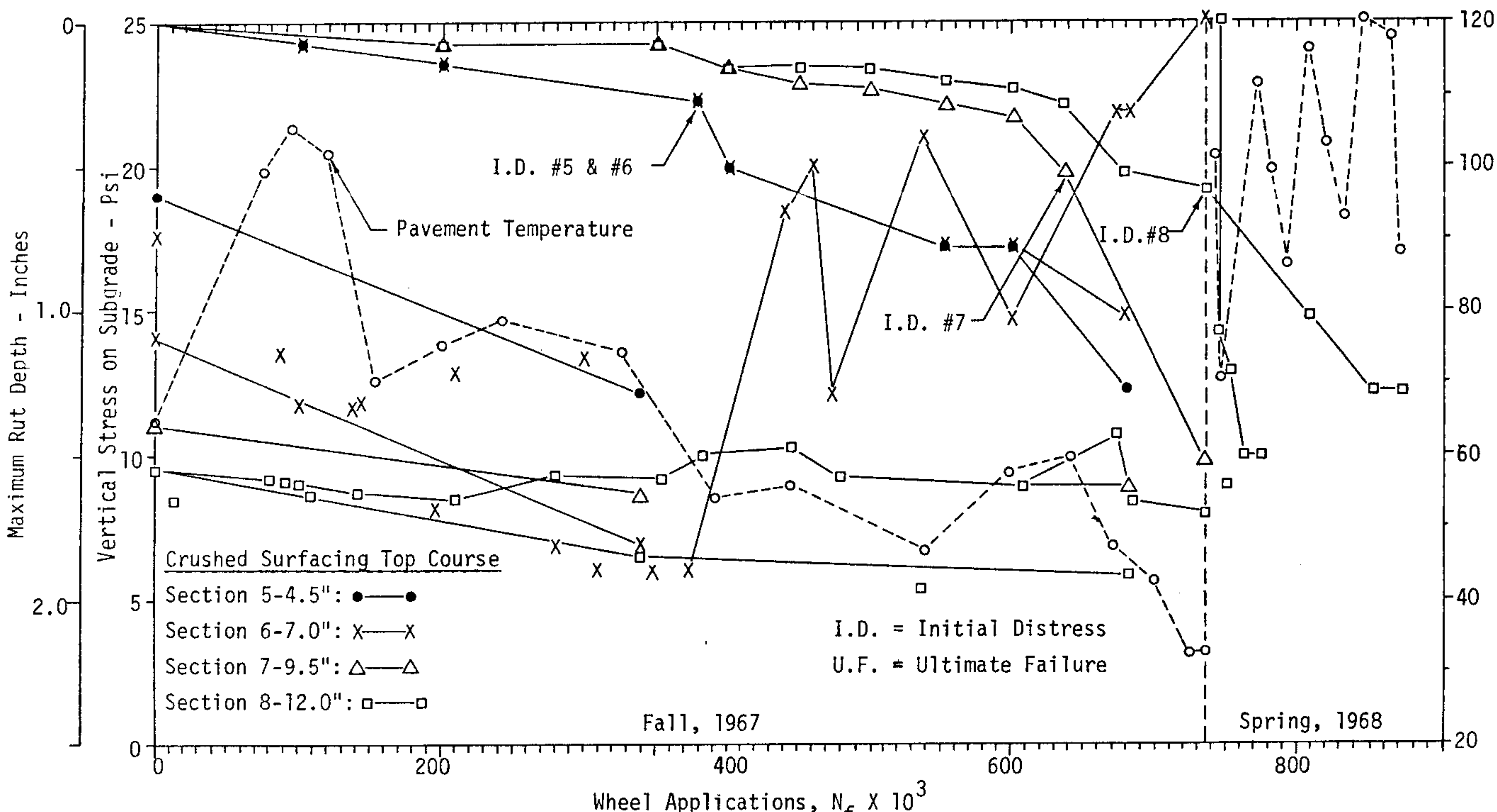


Figure 34: Comparison of Vertical Stress at Top of Subgrade, Rut Depth, and Temperature with Wheel Applications for Pavements with Untreated Base (U.T.B.), Ring #3

in pavement design, of both the elastic and non-elastic phenomena. Figures 37, 38, and 39 for Ring 4 point out that rut depths did stabilize with time, especially for the structurally adequate pavements.

Another important factor influencing rut depth is the vertical stress imposed by wheel loads through the pavement structure to the subgrade. Vesić and Domaschuk [23] show, in Figure 40, the relationship of the rut depth to vertical stress on the subgrade obtained during the 1959 and 1960 studies. This figure shows that a stress level exists beyond which rutting rapidly increases and below which it remains essentially constant, indicating that the distress remained exclusively within the pavement structure. Here the critical vertical stress level seems to lie between 9 and 11 psi, with a slight tendency to increase with the wheel load.

An attempt to try to correlate deflection with rut depth as shown in Tables 9, 10, and 11 and in Figures 19, 20, and 21 was not entirely successful. It appears that a critical maximum rut depth of 0.25 inches was established for the thin pavements as it was this point when these pavements experienced initial distress. Figures 31-39 add the variables of temperature and vertical stress to the rut depth phenomena. Of the two, vertical stress is the most important, thus confirming the findings of Vesić and Domaschuk. All the pavements that failed had vertical stresses at the top of the subgrade higher than 10 psi. In the spring they were often much higher. These vertical stresses combined with environmental factors were probably the cause of punching shear failures in the spring. Rutting was due primarily to compression and distortion of the subgrade soil.

The findings of Vesić and Domaschuk justifies the selection of limiting vertical subgrade stress as one of the major design criteria in flexible pavement design. However, one should note that limiting vertical

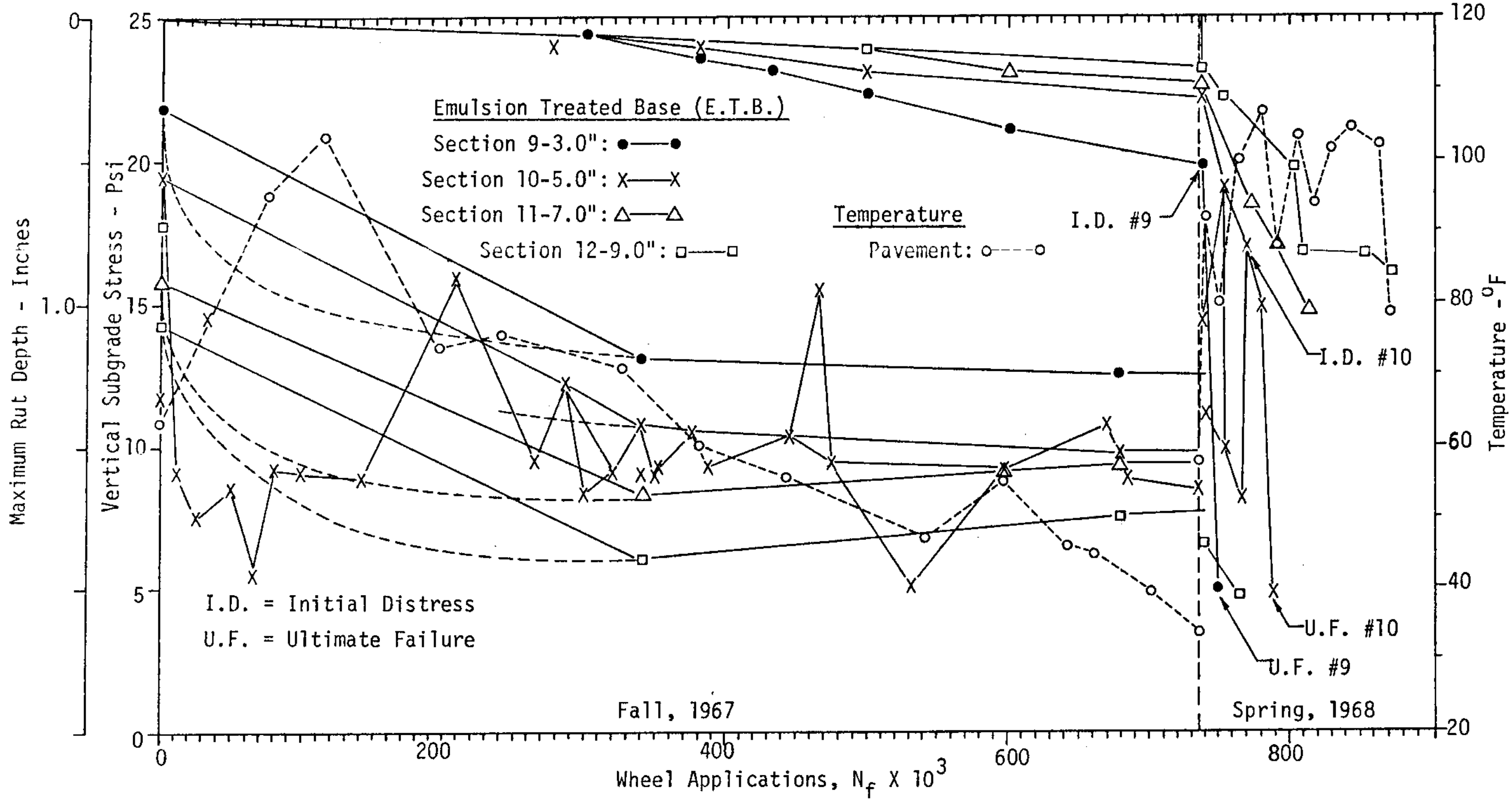


Figure 35: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature with Wheel Applications for Pavements with Emulsion Treated Base (E.T.B.), Ring #3

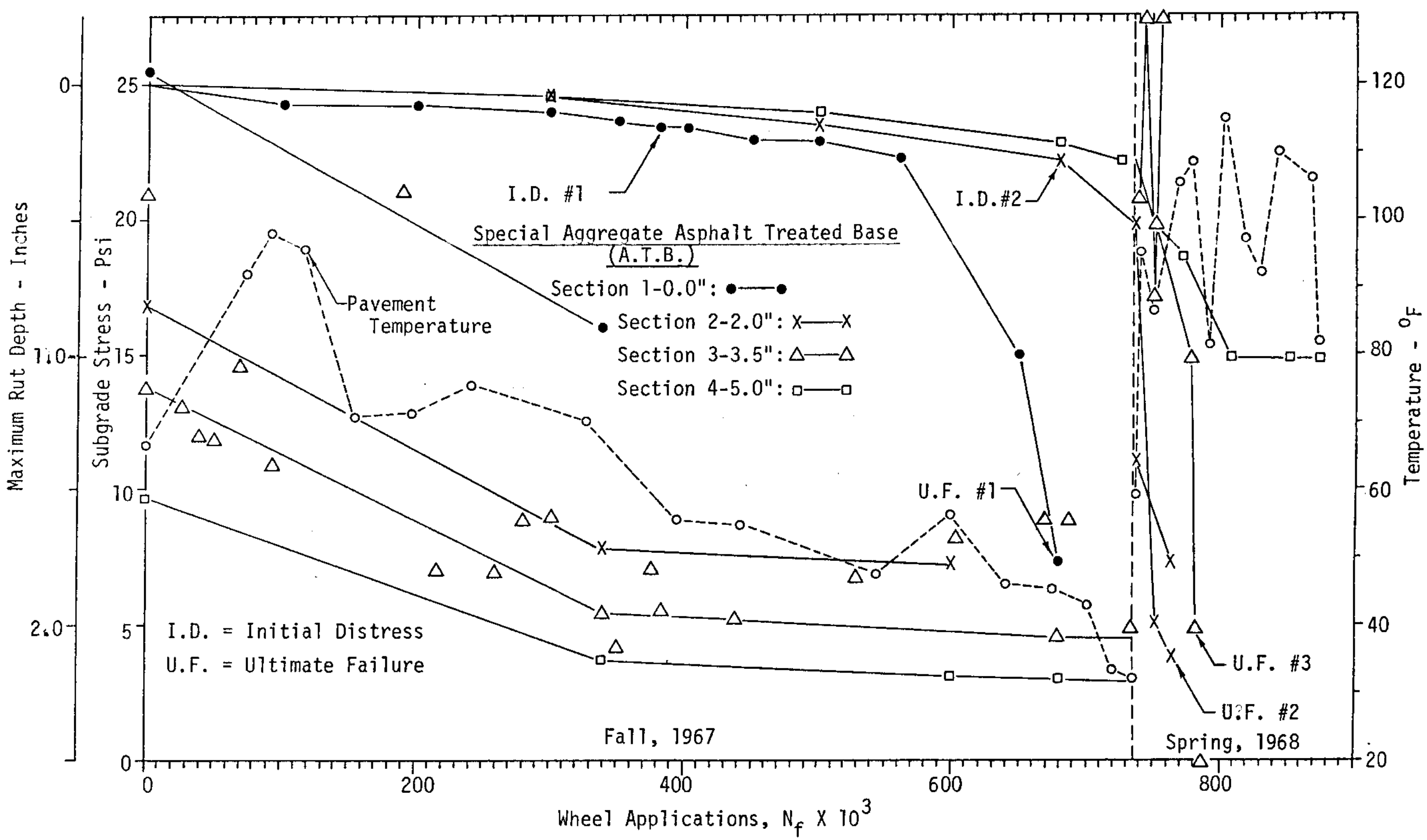


Figure 36: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature with Wheel Applications for Pavements with Special Aggregate Asphalt Treated Base (A.T.B.), Ring #3

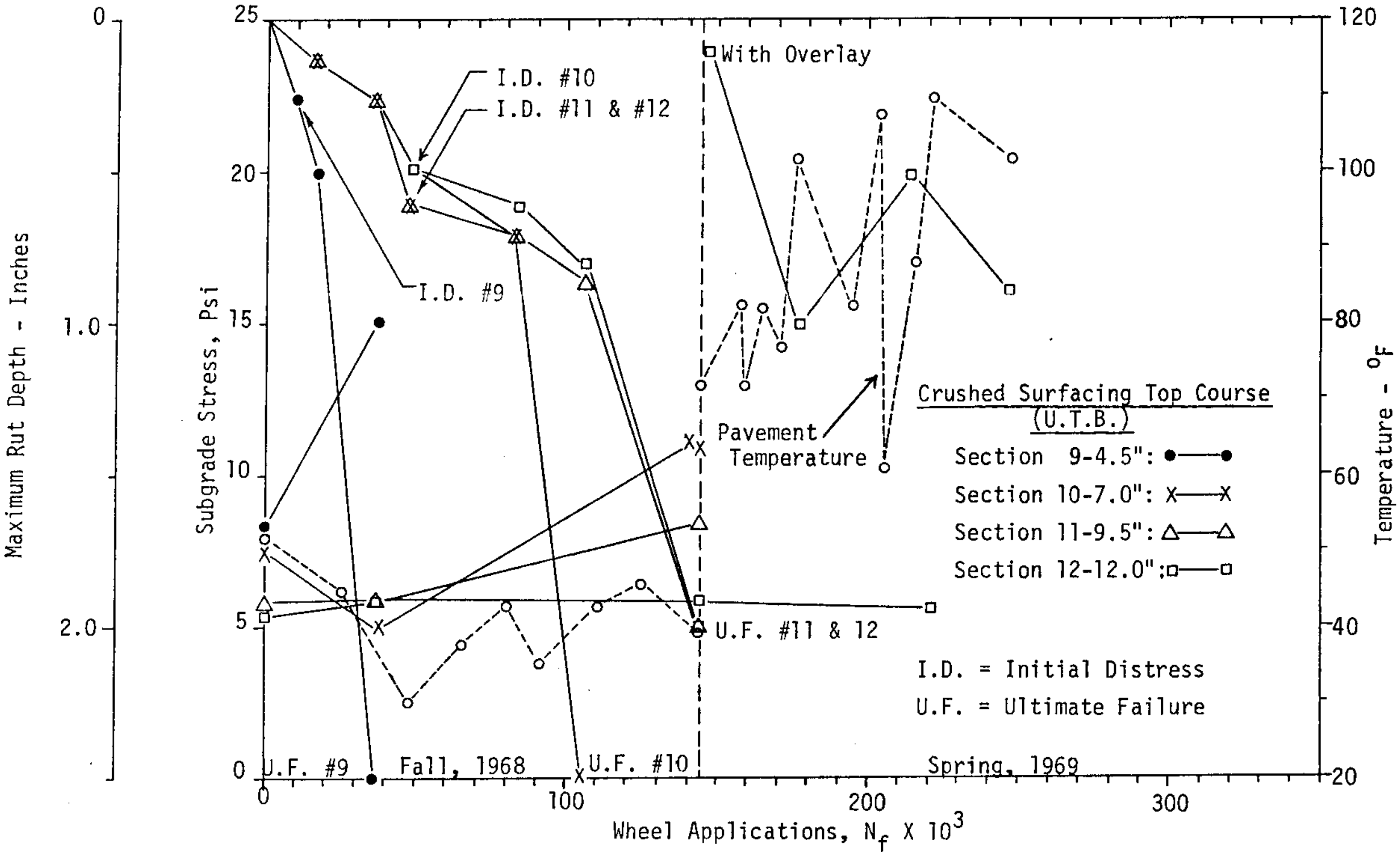


Figure 37: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature with Wheel Applications for Pavement with Untreated Base (U.T.B.), Ring #4

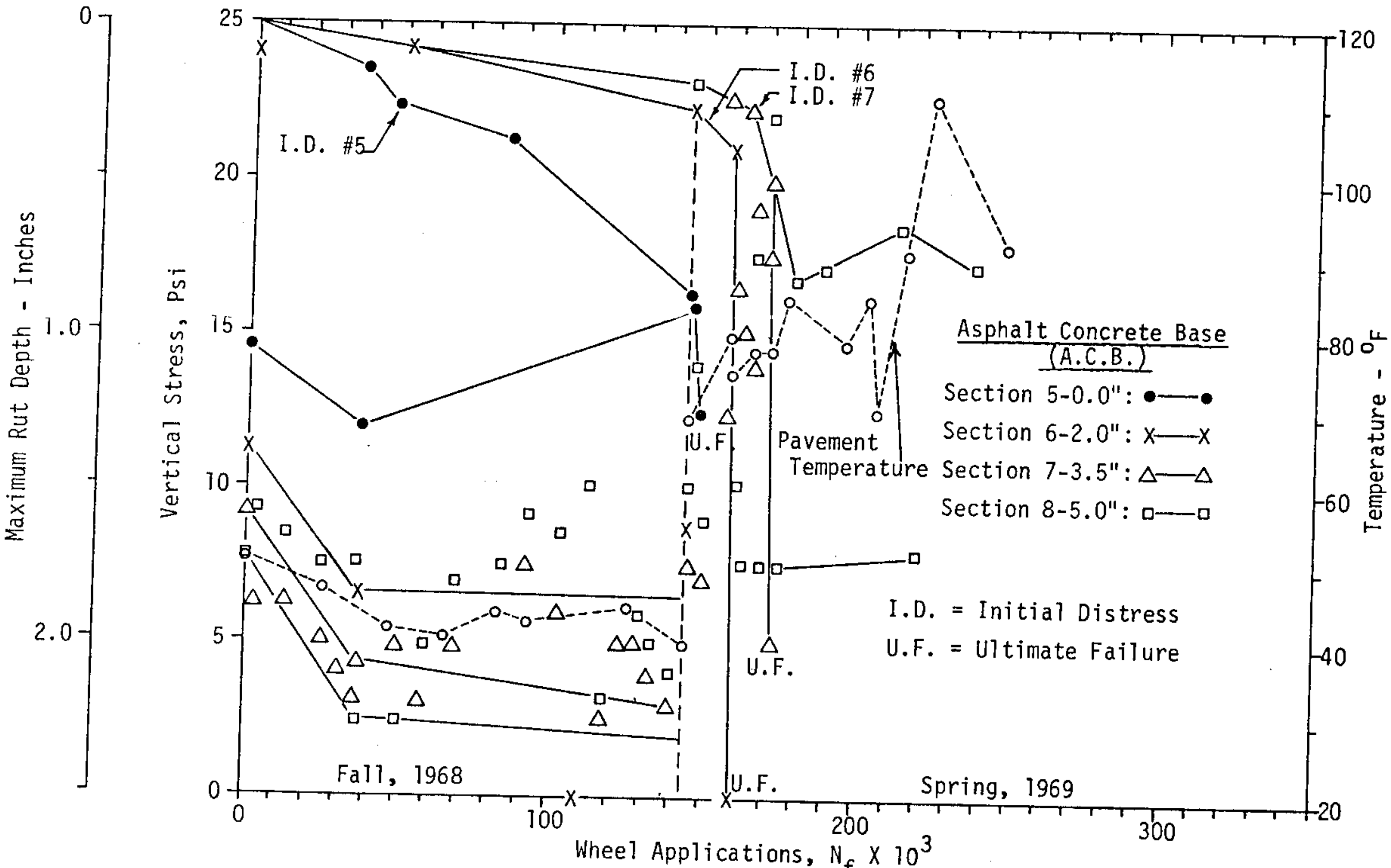


Figure 38: Comparison of Vertical Stress at Top of Subgrade, Rut Depth and Temperature with Wheel Applications for Pavement with Asphalt Concrete Base (A.C.B.), Ring #4

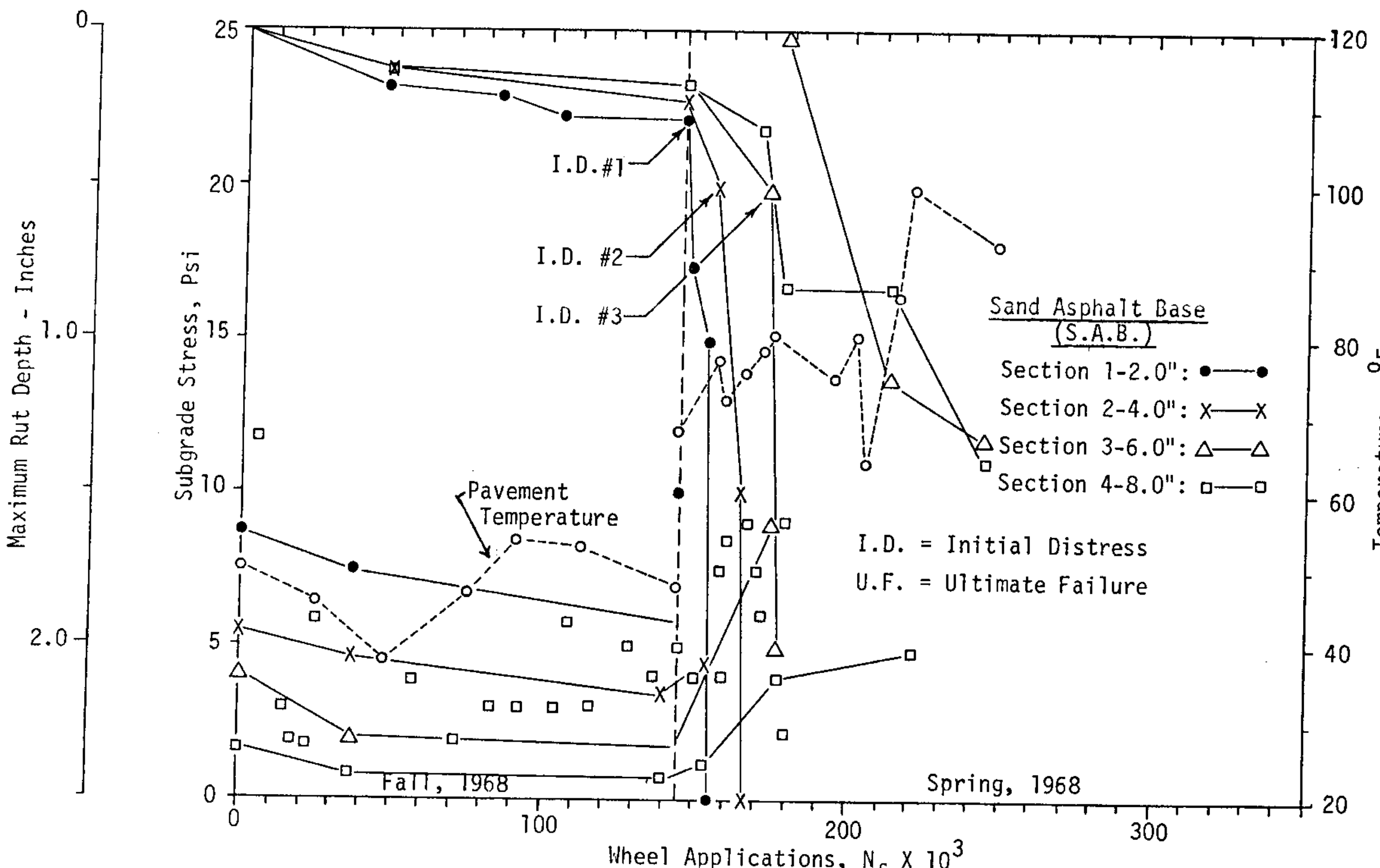
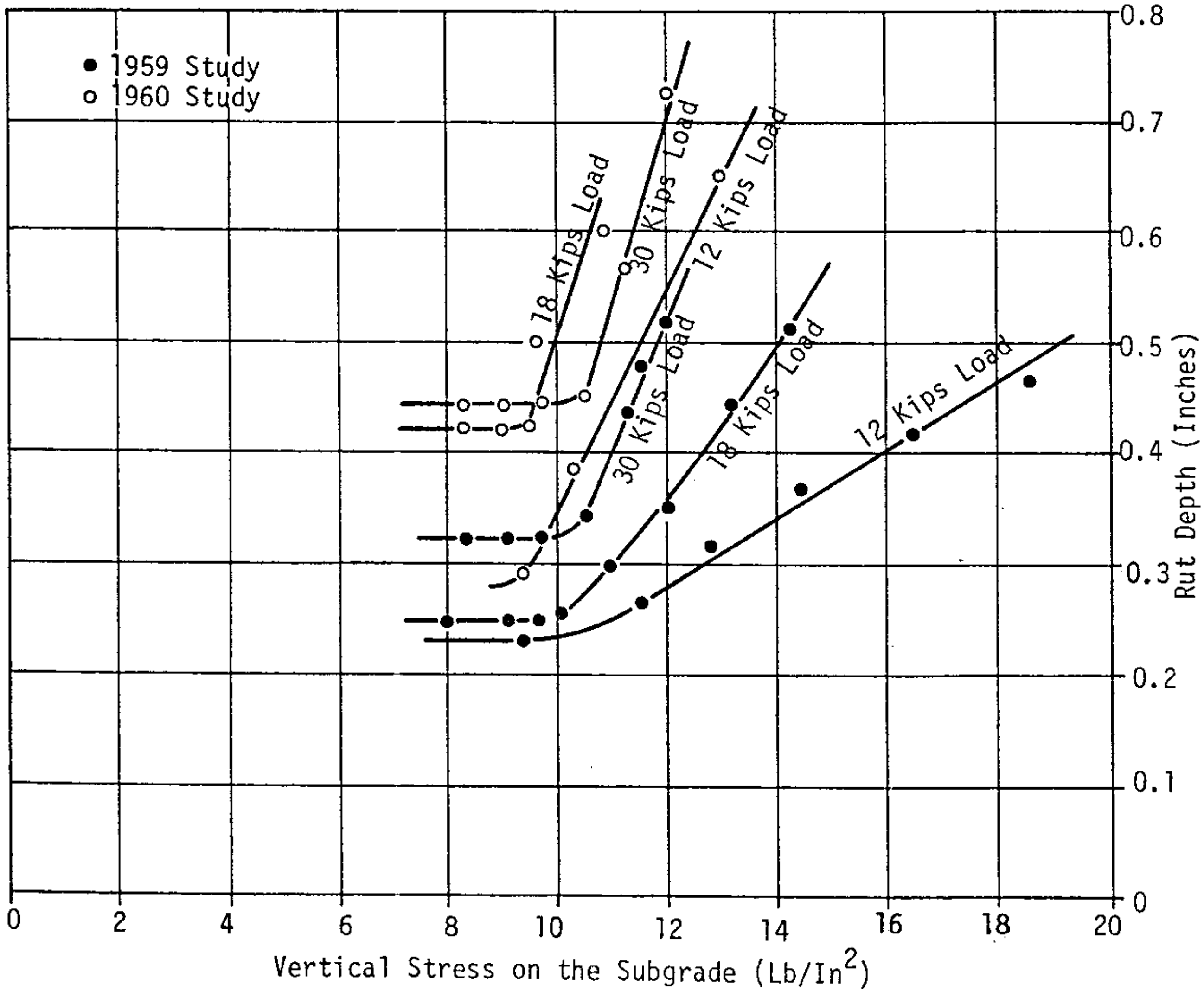


Figure 39: Comparison of Subgrade Stress at Top of Subgrade, Rut Depth and Temperature With Wheel Applications for Pavements with Sand Asphalt Base (S.A.B.), Ring #4



(Reference 23)

Figure 40: Surface Rut Depth as a Function of Vertical Stress on the Subgrade

subgrade stress will not necessarily prevent surface rutting; the latter may be due to other factors.

The Asphalt Institute tried to predict the rutting obtained in Ring 4 using the Moavenzadeh layered viscoelastic computer program. The results obtained were approximately 5 to 10 percent of the measured values [24]. Obviously, more research is needed to explore the use of viscoelastic theory to predict pavement rutting.

Rutting, if it becomes deep enough and uneven, causing a poor ride, can itself be classified as a failure criteria. This has been taken into account in the Present Serviceability Index [5, 19, 21].

The existence of an initial vertical subgrade stress level, which, of course, must be related to the strength and deformation characteristics of the subgrade, partly supports the basic design philosophy of the CBR method, at least for conventionally constructed flexible pavements with untreated bases. This also points out the inability of that method to take into account the better spreading abilities of improved surfacings (plant-mix hot-rolled asphaltic concrete), as well as of bases possessing some tensile strength (bituminous macadam and soil-cement).

One of the problems is trying to explain why most of these sections developed initial distress at approximately the same load repetitions. It is very possible that a combination of thermal and environmental conditions coupled with mechanical distress may have caused the fall failures. In rings #2 and #3, the abnormal amount of precipitation with low temperatures may have added to the saturation of the subgrade. This lowered the subgrade modulus of rupture and, along with high vertical subgrade stresses, may have caused excess strain in the pavement. Temperature differentials may have caused differential stresses in the pavement structure. Krukar and Cook reported in references 2 and 3 that transverse cracks occurred just after a period of cooling weather with heavy rainfall. Although the temperatures were not of the magnitudes during which thermal cracks occur, temperature coupled with continuous mechanical heavy loading probably accelerated the cracking.

In the spring, no doubt environmental conditions contributed to the sudden punching shear failures. This phenomena has been reported on by Krukar and Cook in references 2, 3, and 4.

Another design parameter which definitely should be taken into account is environment. The longevity of Ring 3 points out this importance. Construction techniques, uniformity and compaction of the subgrade, uniformity of the pavement and other factors must also be considered. These factors are the responsibility of the field engineer. The environmental factors for each area probably should be calculated in the pavement design.

Last but not least, the findings show that untreated base does not add proportionally, inch for inch, the same strength as a treated base. Using thick untreated bases will not add significantly more strength than a thin untreated base. A pavement design would be improved through the use of a thicker treated base.

EVALUATION OF FIELD EQUIVALENCIES

The equivalency concept is a procedure in which a certain thickness of one material may be converted into an equivalent thickness of another material. Many of these equivalency or substitution factors have been developed empirically from field observations and test roads such as WASHO and AASHO. Field observations made at the WSU Test Track have been developed into a series of equivalencies and are shown in Tables 19, 20, and 21. These can be used by the highway engineer to calculate substitution. Use of these equivalencies

Table 19
FIELD EQUIVALENCIES IN TERMS OF
CLASS "E" ASPHALT CONCRETE BASE, RING 1
(4.25 inches of Class "B" Asphalt Concrete Wearing Course)

Type of Base	Artificial Saturated Subgrade Conditions[2]		Normal Conditions[3]
	Initial Cracking (inches)	At Failure (inches)	At End of Test (inches)
Class "E" A.C.[1]	1.00	1.00	1.00
Asphalt Treated Base[1]	0.93	1.00	1.00
Cement Treated Base	1.95	1.90	1.50

[1] Hot-mix
[2] Water was artificially introduced
[3] Under all environmental conditions

Table 20

FIELD EQUIVALENCIES ON RINGS #2, #3, AND #4

Type of Base	Conditions	
	Fall[1] Inches	Spring[2] Inches
Crushed Surfacing Top Course (U.T.B.)	9.5	12.0
Emulsion Treated Crushed Surfacing Top Course (E.T.B.)	7.0[3]	9.0
	3.0	9.0
Special Aggregate Asphalt Treated (A.T.B.)[4]	2.0	5.0
Class "F" Asphalt Concrete (A.C.B.)[5]	2.0	5.0
Sand-Asphalt Base (S.A.B.)[6]	2.0	8.0

[1] The thinnest sections which survived this period
[2] The thickest sections which failed during this period
[3] E.T.B. was relatively uncured
[4] Hot-mix with 3.0% asphalt
[5] Hot-mix with 5.8% asphalt
[6] Hot-mix with 5.2% asphalt

Table 21

FIELD EQUIVALENCY FACTORS IN TERMS OF
CRUSHED ROCK BASE (U.T.B.) AND ASPHALT CONCRETE BASE (A.C.B.)

Type of Base	In Terms of U.T.B. Conditions		In Terms of A.C.B. Conditions	
	Fall[1] Inches	Spring[2] Inches	Fall[1] Inches	Spring[2] Inches
U.T.B.	1.00	1.00	4.75	2.40
E.T.B.	0.74[3]	0.75	3.50[3]	1.80
	0.21[4]	0.75	1.50[4]	1.80
A.T.B.[5]	0.21	0.42	1.00	1.00
A.C.B.[6]	0.21	0.42	1.00	1.00
S.A.B.[7]	0.21	0.67	1.00	1.00

[1] This can be assumed to be under good environmental conditions
[2] Environmental conditions were poor
[3] E.T.B. was relatively uncured
[4] E.T.B. was estimated to be at least 50% wind
[5] Hot-mix with 3.0% asphalt
[6] Hot-mix with 5.8% asphalt
[7] Hot-mix with 5.2% asphalt

raises questions of validity of the values, confidence in their use, and need for modification.

Table 19 shows the equivalencies obtained from Ring 1. Although the asphalt contents of the class "E" and asphalt treated bases were approximately the same, the latter contained inferior aggregates, in that they were non-fractured. In a longer run test, the equivalency factor between the two may show the class "E" asphalt concrete base to be slightly superior to the asphalt treated base with screened aggregates. The equivalency for the cement-treated base was 1.50. This was a low grade cement-treated base. This was much higher than that obtained from the AASHO Road Test [5, 25, 25]. Here 1-inch of asphalt concrete base was equal to 1.3 inches of cement treated base. Therefore, it is very likely that the WSU equivalency factor for C.T.B. was too low. The equivalency factor obtained by inducing failure by saturating the subgrade was perhaps more realistic for the C.T.B. Excavation of the C.T.B. revealed that it was badly fatigue cracked in the wheel path, and thus its equivalency factor should be lowered.

Field equivalency factors obtained from Rings 2, 3, and 4 for two different environmental conditions are shown in Tables 20 and 21; the latter is based in terms of crushed rock base and asphalt concrete base. The validity of these factors is examined by comparing them with results obtained elsewhere. At the AASHO Road Test, one inch of bituminous surface was equivalent to 3.0 inches of crushed stone base or to 4 inches of sand-gravel subbase [5]. Experience in Canada indicates that the ratio of the relative supporting capacity of bituminous concrete surface to granular base may be as low as 2 to 1 (20). Shook and Finn [27], using AASHO Road Test data, showed that larger equivalencies of asphalt-concrete surfacing in terms of crushed rock base ranged from 2 to 6.7 depending upon the criteria used. The Asphalt

Institute [28] recommends using substitution ratios of 2.0 to 2.7 inches of untreated granular base for 1.0 inch of asphalt layer depending upon the quality of the granular base. The equivalency factors between the untreated crushed rock and the asphalt concrete base appear to be reasonable.

However, one should ask how valid are the equivalency factors between the emulsion treated base and the asphalt concrete base. Table 22 [29] shows that the WSU equivalency factors between these materials may be too high. The problem with emulsion treated bases is that they require time to cure. It may take as long as two years before they reach their full strength [30], and depending upon their curing time, their strengths will vary. For example, at placement their equivalency is equal to that of untreated base. The emulsion treated base for the full period in Ring 2 was relatively uncured, and hence its equivalency factor was high compared to the asphalt concrete base. The Bitumuls Base Treatment Manual [31] shows in Table 23 that depending upon what kind of subgrade one has and at what period one has assumed the emulsion treated base to be, one will obtain different equivalency factors. Therefore, it can be said that under WSU conditions of test, the equivalency factors between the emulsion treated base and the asphalt concrete base are valid. However, the highway engineer should decide which assumptions are valid for his highway.

The WSU equivalency factors between the low asphalt content treated base and the class "F" asphalt concrete base were the same (Tables 20 and 21). The asphalt treated base with a low asphalt content and uncrushed rock aggregate had a comparatively good performance record. It is possible that in a longer run test and perhaps different conditions, this material will not stand up as well as the asphalt concrete. Terrel and Awad [32] appear to come to this conclusion. Therefore, it is suggested that these field equivalencies be adjusted in favor of the asphalt concrete base, by perhaps 1.3 to 1.

Table 22

THICKNESS FACTORS FOR PLANT-MIXED ASPHALT BASE

Asphalt Base	Thickness Factor, f
Asphalt concrete	1
Hot-mix sand asphalt	1.3
High-quality, well controlled, well graded aggregate, but using cutback or emulsified asphalt	1.4
Other mixed using cutback or emulsified asphalt	1.4

(The Asphalt Institute [29])

Table 23

TYPICAL THICKNESS DESIGNS FOR BITUMULS TREATED BASES
(3 inch Asphalt Concrete Surfacing)

Subgrade Classification	Class of Traffic, DTN[1]	Aggregate Base Inches[1]	BITUMULS TREATED BASE M_R= 50,000 psi Inches	M_R= 150,000 psi Inches	M_R= 250,000 psi Inches	Asphalt Concrete Base Inches[1,2]
Poor	1	7	6	5	4	3 1/2
	10	12	9	8	7	6
	50	16	12	10	9	8
	100	18	14	12	11	9
Fair	1	5	4	3 1/2	3	2 1/2
	10	11	9	7	6	5 1/2
	50	14	12	9	8	7
	100	16	13	11	9	8
Good	1	4	3 1/2	3	2 1/2	2
	10	8	7	6	4 1/2	4
	50	11	10	8	6	5 1/2
	100	13	12	9	7	6 1/2

[1] The Asphalt Institute--Thickness Design Manual (MS-1).
[2] The asphalt concrete base in similar to the high quality Type IV mix described in the Asphalt Institute Specifications and Construction Methods for Asphalt Concrete and Other Plant Mix Types (SS-1).

Table 23 refers to the subgrade as "Poor," "Fair" and "Good." In most cases road building agencies will have tests to describe the strength properties of the subgrade,and engineers will have criteria suitable to evaluate these descriptive terms. However, some further explanation may be necessary as follows:

1. Poor subgrade soils would include the highly plastic A-5, A-6, and A-7 soils (AASHO classification). In general, these subgrade soils would have a Resistance R Value of less than 12.

2. Fair subgrade soils would include the medium plasticity A-4, A-5, A-6,and A-7 soils. In general, these subgrade soils would have a Resistance R Value between 12 and 30.

(Reference 31)

The equivalency factor for the sand-asphalt base appears to be equal to that of asphalt concrete base. The Asphalt Institute [29] suggests that this ratio should be 1.3 to 1 and that 1.00 inch of sand-asphalt base is equal to 1.8 inches of crushed rock base (33). From this it appears that the WSU equivalency factors for the sand-asphalt base should be increased to the value of 1.3, similar to that recommended by the Asphalt Institute, as in Table 22.

These equivalency factors should be used with care and judgment. Different assumptions and conditions may require that the equivalency factors be adjusted. This will depend upon the highway engineer and his value judgment data and criteria. Table 24 shows the adjusted field equivalency factors for the various materials used at the test track.

SUMMARY AND CONCLUSIONS

A pavement structure is a very complicated system that is difficult to explain in terms of performance. It is influenced by many variables such as temperature, moisture, traffic, construction practices, as well as material variations. The highway engineer has attempted to explain this complicated performance in terms of simple material responses under ideal, or at least constant, conditions. Such tests as the California Bearing Ratio (CBR), Bearing Value Determination (Plate Bearing Test), and Hveem Resistance Value (R) Method, were devised to solve this complicated problem. Each has worked well in the past. However, the costs of constructing or reconstructing roads are increasing every year. Materials are becoming more scarce. The engineer must find better methods of building and maintaining roads to last the number of years intended. No longer can materials be wasted. Testing of new ideas and methods must continue to insure the best possible solutions are used to build roads. The WSU test track is a vital link in this research.

Many problems did exist in obtaining data from Test Rings 1, 2, 3, and 4, but these problems have helped to illustrate the complexities of a pavement system. Technology has not advanced to the point where these complexities can be effectively handled. Therefore, some of the variables inherent in the test track must be eliminated or, at least, minimized.

Recommendations made earlier in this report are again listed for emphasis:

1. The test track should be enclosed in a building so that the temperature and moisture can be controlled.
2. French drains and diversionary ditches should be placed around the track to help control the moisture of the subgrade.
3. A horizontal concrete shelf should be constructed under the test ring to reduce the effect of the varying subgrade depth.
4. Load wheels should have different length radial arms so that the loads are staggered on the pavement, and thereby more closely simulate actual traffic conditions. This may be more economically accomplished by modifying the eccentricity mechanism to speed up the wheel load coverage. These modifications to the physical plant should be done before a new test ring is constructed.
5. The driving mechanism and the shock system should be modified to take advantage of the speed capability of the apparatus.

An attempt was made to explain the performance of the test sections in terms of the parameters measured by the Hveem method and the elastic layer theory. Each method will be discussed separately.

The Hveem method primarily is concerned with designing a pavement structure that will resist excessive permanent deformation during its intended life. The actual number of wheel loads necessary to cause a rut depth of 0.25 inches in each test section was recorded and compared to the design charts presently used by the State of Washington. The correlation was very poor. This poor correlation could have been caused by many variables; one being the difficulty in knowing the actual R-value during the life of the pavement. If the subgrade had been compacted to its equilibrium condition initially, perhaps more firm conclusions would have been possible. The compaction properties of the Palouse silt would prevent this approach.

A comprehensive study of the test track sections using the elastic layer theory was conducted by Kingham and Kallas [7]. Their conclusions are listed below:

1. Laboratory-fabricated test specimens for fatigue tests did provide the same results as those cored from untraveled portions of the test pavements. This finding suggests that lab fabrication can produce a specimen representative of the field.
2. Predictions from the stress-controlled laboratory tests were very close to those observed for 3 of 6 full-depth asphalt test sections.
3. Stress controlled laboratory fatigue tests predicted the life of full-depth asphalt pavements better than strain controlled tests. Both stress modes tended to over-predict full-depth asphalt pavement life.
4. Field failure criteria developed from AASHO Road Test data gave slightly closer predictions than laboratory results. Promise exists for the application of the criteria to asphalt mixtures that have different dynamic modulus-temperature relationships than the AASHO Road Test bituminous base.

5. Neither the laboratory fatigue data nor the field failure criteria predicted satisfactorily the lives of the crushed stone base test sections.

As discussed earlier, construction variables have a great influence on the performance of the pavement structure. It is imperative that the materials used be as consistent as possible. The subgrade should be compacted at the densities normally found in the field. The asphalt mixtures should be consistent and agree with the mix design. The fact that much care and inspection was spent during the construction of the test track rings only points out the difficulties that highway engineers have in the field with inspection, construction and quality control.

Design parameters such as deflections, strains, stresses, and rut depths were examined as to their role in flexible pavement design. Deflections from the test track were too high to be used for design purposes and bear out the ASSHO and CGRA findings that pavement deflections of less than 0.050 inches cause no failures. It was found that there are no "critical" stress levels because if they are increased, pavement life is accordingly reduced and vice-versa. There appears to exist critical vertical subgrade stresses which will cause rutting and failure. This level is between 9 and 11 psi and for design perhaps should be less than 5 psi. Rut depths under WSU conditions seem to be correlated to densification, shoving, and vertical subgrade stress levels. For thin pavements, a rut depth of 0.25 inches appears to be critical under the right environmental conditions. More important than anything else is environment and then construction and quality. These will increase pavement life.

The field equivalencies obtained from the four rings were examined and were compared with other studies. Some were adjusted to take into account other experience, and new adjusted field equivalency factors were developed.

As highway engineers begin to more fully understand the problems, the complexities normally inherent in a pavement structure can be reintroduced into the test track pavement one at a time. Only in this way can one document what is known and realized what is unknown in order to solve the most complicated and interesting problem facing the highway engineer today.

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APPENDIX A

CALCULATION OF R-VALUES FOR RINGS 2, 3 AND 4

UTB--Ring 2

Wheel Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
0	14.6	55	18.0	53	15.4	55	14.8	55
25,000	15.5	55	19.0	51	16.4	55	15.6	55
50,000	16.7	54	19.5	48	16.9	54	16.4	55
75,000	17.0	53	20.5	43	17.3	53	16.9	54
100,000	18.0	53	22.0	25	17.8	53	17.6	53
125,000	18.3	52					18.7	51
150,000	18.7	52					19.2	51
175,000	19.0						19.9	47
Average R-Value		53.4		44		54		53

UTB--Ring 3

Wheel	5		6		7		8	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
100	16.5	55	15.2	55	15.2	55	13.7	55
72,400	16.5	55	16.0	55	15.2	55	14.7	55
92,600	16.6	55	16.3	55	15.6	55	14.7	55
116,800	16.5	55	16.4	55	15.6	55	14.9	55
152,800	16.9	54	16.8	54	15.8	55	15.2	55
199,400	17.0	54	17.0	54	16.0	55	15.6	55
240,900	17.3	54	17.2	54	16.4	55	15.9	55
326,500	17.6	53	17.4	54	16.5	55	16.2	55
396,200			17.3	54	16.6	55	16.8	54
442,400					17.5	54	17.6	53
543,500					20.0	47	18.8	51
595,800					21.5		19.2	50
640,200							19.5	
Average R-Value		54		54		51.6		53

UTB--Ring 4

Wheel	9		10		11		12	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
400	21.4	35	19.2	50	19.3	50	18.4	52
25,000	26.3	3	19.2	50	19.4	50	19.2	50
Average R-Value		35		50		50		51

ETB--Ring 2

Wheel	5		6		7		8	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
0	15.9	55	15.7	55	15.6	55	15.0	55
25,000	17.0	54	16.7	54	16.5	54	15.7	55
50,000	18.4	52	17.9	53	17.0	54	16.7	54
75.000	20.0	47	18.5	52	18.0	53	17.1	54
100,000	22.0	26	21.0	39	18.7	52	18.2	52
125,000					19.3	49	18.7	52
150,000					20.0	47	19.3	49
175,000					20.5	43	19.0	51
205,425					21.0	39	19.8	47
Average R-Value		46.8		50.6		49.6		52.1

ATB--Ring 2

Wheel	9		10		11		12	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
0	15.8	55	15.2	55	15.6	55	16.0	55
25,000	16.8	54	16.5	55	16.0	55	16.5	55
50,000	17.8	53	17.5	53	17.2	54	16.7	54
75,000	18.0	53	18.2	52	17.3	54	16.8	54
100,000	18.5	52	18.4	52	18.0	53	17.0	54
125,000	19.0	51	18.8	51	18.5	52	17.5	53
150,000	19.5	48	19.4	48	19.0	51	18.0	53
175,000	20.0	47	20.2	46	19.6	48	19.0	51
205,425	21.0	38	20.8	40	20.2	46	19.8	47
Average R-Value		50.1		50.2		52		46.9

ATB--Ring 3

Wheel	1		2		3		4	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
100	14.3	55	15.0	55	14.5	55	14.3	55
72,400	16.0	55	16.4	55	16.2	55	15.3	55
92,600	16.3	55	16.3	55	16.2	55	15.8	55
116,800	16.5	55	16.6	55	17.1	54	15.9	55
152,800	16.0	55	15.8	55	17.0	54	16.5	55
199,400	16.3	55	16.0	55	17.4	54	16.6	55
240,900	16.6	55	16.6	55	17.6	53	16.7	54
326,000	17.2	54	16.8	54	19.0	51	17.2	54
396,200	18.9	51	18.0	53	19.0	51	17.3	54
442,400	18.9	51	18.3	52	18.2	53	17.6	53
543,500	18.9	51	18.5	52	19.3	50	19.2	50
595,800	19.0	51	18.6	52	19.6	49	19.6	49
640,200			19.0	51	19.5	49	19.5	49
735,650			21.8		21.0		21.0	
Average R-Value		53.25		53.29		51.9		52.5

ACB--Ring 4

Wheel	5		6		7		8	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
400	17.7	53	17.7	53	17.7	53	17.0	54
25,000	17.6	53	17.7	53	17.7	53	17.7	53
49,100	17.8	53	18.1	53	18.2	53	17.9	53
91,200			18.6	52	18.4	52	18.2	53
143,400			26.0	3	25.0	7	28.0	3
157,000					27.0	3	27.5	3
158,200					27.8	3	27.6	3
164,800					28.3	3	27.0	3
Average R-Value		53		36		34		34

SAB--Ring 4

Wheel	1		2		3		4	
Loads	N%	R-Value	N%	R-Value	N%	R-Value	N%	R-Value
400	17.5	54	17.4	54	17.3	54	17.2	54
25,000	17.7	53	17.5	54	17.4	54	17.3	54
49,100	18.0	53	17.5	54	17.5	54	17.4	54
91,200	18.2	53	17.6	53	17.6	53	17.5	54
143,400			27.0	5	27.0	5	25.0	5
157,000			27.5	5	27.5	5	24.0	10
158,200			27.6	5	27.8	5	25.1	7
164,800					27.6	5	24.8	7
Average R-Value		53.2		39.7		39.7		40