EVALUATION OF OKLAHOMA PAVEMENT DESIGN PROCEDURES

Final Report

REPORT NO. 84-60

Submitted to:

MR. C. DWIGHT HIXON, P.E.
RESEARCH ENGINEER
STATE OF OKLAHOMA DEPARTMENT OF TRANSPORTATION
200 N.E. 21ST STREET
OKLAHOMA CITY, OKLAHOMA 73105

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ARE INC

ARE Inc-ENGINEERING CONSULTANTS
2600 DELLANA LANE · AUSTIN, TEXAS · 78746

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John F. Nixon, Jobaid Kabir, Frank McCullough, Fred Finn

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2600 Dellana Lane
Austin, Texas 78746

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16. Abstract

This report covers the results of Phases II and III of the research project. Based upon the findings of this study, recommendations were made for possible improvement of the currently used design and management procedures. Specific recommendations were made for improved material specifications to avoid stripping in the pavement structure as this was found to be the most important reason for the failure of pavements. A research plan has been proposed for future research.

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Research Engineer
Oklahoma Department of Transportation
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By

ARE Inc - Engineering Consultants
2600 Dellana Lane
Austin, Texas 78746

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EXECUTIVE SUMMARY OF PHASE I

This report presents the results of the administrative and technical investigations carried out in Phase-I of this pavement evaluation study. Preliminary findings of this report are based on numerous meetings and discussions with the Oklahoma Department of Transportation (ODOT) personnel, diagnostic evaluations, laboratory investigations, nondestructive field testing and evaluation. This investigation included rigid, flexible, and composite pavement sites from different locations of the Oklahoma highway system to cover a broad range of climatic and geological conditions.

Irrespective of pavement type, the majority of the failures are occurring due to material problems in the asphalt concrete mixtures in either surface or base layers. Moisture susceptibility of the mixtures used in the base and surface layers is mainly responsible for the asphalt stripping occurring from the aggregates. Shear failure of an underlying layer caused by stripping is in turn responsible for rutting, shoving, and cracking in flexible pavements, faulting in rigid pavements, and rutting, shoving, and reflection cracking in the composite pavement.

The preliminary recommendations are to re-evaluate the A.C. mix design requirements in terms of moisture susceptibility and higher load carrying capacity. Some type of load transfer between the slabs and proper joint seal should be considered for the rigid pavement sites. The composite pavement should be provided with a stress relieving layer (fabric, asphalt-rubber or open graded mix) for reducing reflection cracking in addition to improvement of the mix design requirements regarding stripping and load carrying capacity.

A detailed review of the ODOT pavement design and management practices is being conducted by ARE Inc to produce any recommendations for change. Also comparisons will be made between the ODOT design method and the revised AASHTO pavement design guides currently being developed. On the basis of these comparisons and investigative evaluations of eight Oklahoma pavement sites, final recommendations will be made and submitted to ODOT in the final report.

EXECUTIVE SUMMARY OF PHASES II AND III

The results of technical investigations carried out in Phase-II and III of the research project entitled "Evaluation of Oklahoma Pavement Design Procedures" are presented in this report. A detailed review of the Oklahoma Department of Transportation (ODOT) design and management practices, revised AASHTO pavement design guide, along with the Phase-I report of this project lead to the recommendations made herein. These recommendations are based on the study conducted for the eight pavement sites selected by the ODOT. These sites include rigid, flexible and composite pavements from different locations within the Oklahoma highway system to cover a broad range of climatic and geological conditions.

Based upon the findings of this study, several recommendations were made for possible improvement of the currently used design procedures. Recommendations were made for improved material specifications to avoid stripping in the pavement structure as this was found to be the most important reason for the failure of pavements. Other recommendations include: (i) new material characterization tests so that deflection measurements can be used to predict properties; (ii) use of 18 kip EAL as the design load, and (iii) use of reliability concepts in the design. Recommendations were also made for joint management and load transfer devices.

The recommendations developed from this study for upgrading currently used design procedures, pavement management, and material specifications are qualitative in nature. Further investigations are required to incorporate these recommendations into current practices. A plan has been proposed for future research.

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CHAPTER 1
INTRODUCTION

Pavement thickness design procedures (4) currently practiced by the Oklahoma Department of Transportation (ODOT), were implemented in 1962. This was last revised in 1965. Since that time, the available technology for pavement design has advanced substantially due to the availability of high speed computers. Also, during this period the traffic volume and various other related design parameters have changed.

The ODOT has observed over the past years that some pavements, including both rigid and flexible, have shown premature distress over a wide range of climatic and geologic conditions within the state. To take advantage of the available advanced technology and changed design parameters, the ODOT felt it necessary to review and possibly update the currently practiced pavement design procedures and management practices. It is anticipated that updating of the ODOT design and management procedures will provide a full design life of the pavement with a minimum of maintenance.

Eight representative pavement sites were selected by the ODOT for investigation in this research project. These sites were selected throughout the state to cover a wide range of climatic and geological conditions. The sites were selected to include failed and good condition pavements of these types: rigid, flexible and composite. A detailed description of these sites is given in Table 1 and are shown in Figure 1. The reader is referred to (3) for all the data of Phase-I for this study, as this data will not be included in this report.

Table 1. Description of pavement projects selected for evaluation.

Site No.	Project No.	Highway	County	Date of Design	Surface Type	Years in Service
1	I-40-4(50)127	I-40	Canadian	2-24-69	PC	17
	From 2 1/2 miles west of S.H. 92 in Yukon west approx. 7 3/4 miles, just past the U.S. 81 Interchange.					
2	F-DP-186(115)	U.S. 69	Pittsburg	3-31-81	AC	2
	From the U.S. 270 Interchange in McAlester north approx. 5 miles to S.H. 113.					
3	I-40-1(16)000	I-40	Beckham	6-27-66 7-7-72	AC	11
	From 1/4 mile east of the S.H. 30 Interchange in Erick west approx. 7 1/2 miles to the Texas State Line.					
4	SAP-3(121)	U.S. 69	Atoka	10-22-80	AC	3
	From south of Caney north approx. 7 miles north of Tushka.					
5	FAP-F-186(77)	U.S. 69	McIntosh/ Muskogee	3-12-73	PC	3
	From the north Checotah Interchange (w/old U.S. 69) north approx. 5 miles to the Oktaha Interchange (w/old U.S. 69).					
6	I-35-2(89)082	I-35	McClain	10-30-69	AC	15
	From 1/2 mile north of the S.H. 59 Interchange (2 miles west of Wayne) south approx. 5 miles to the McClain-Garvin County Line.					
7	FAP-F-481(25) SAP-74(33) pts. I & II	U.S. 75	Washington	9-19-77	AC	6
	From north of Copan, 6 miles south of the Kansas State Line, north approx. 5 miles.					
8	I-35-4(103)193	I-35	Noble		AC	4
	Resurfacing Project - From a pt. 6 miles north of the U.S. 64 Interchange in Perry and extends north approx. 11 miles.					

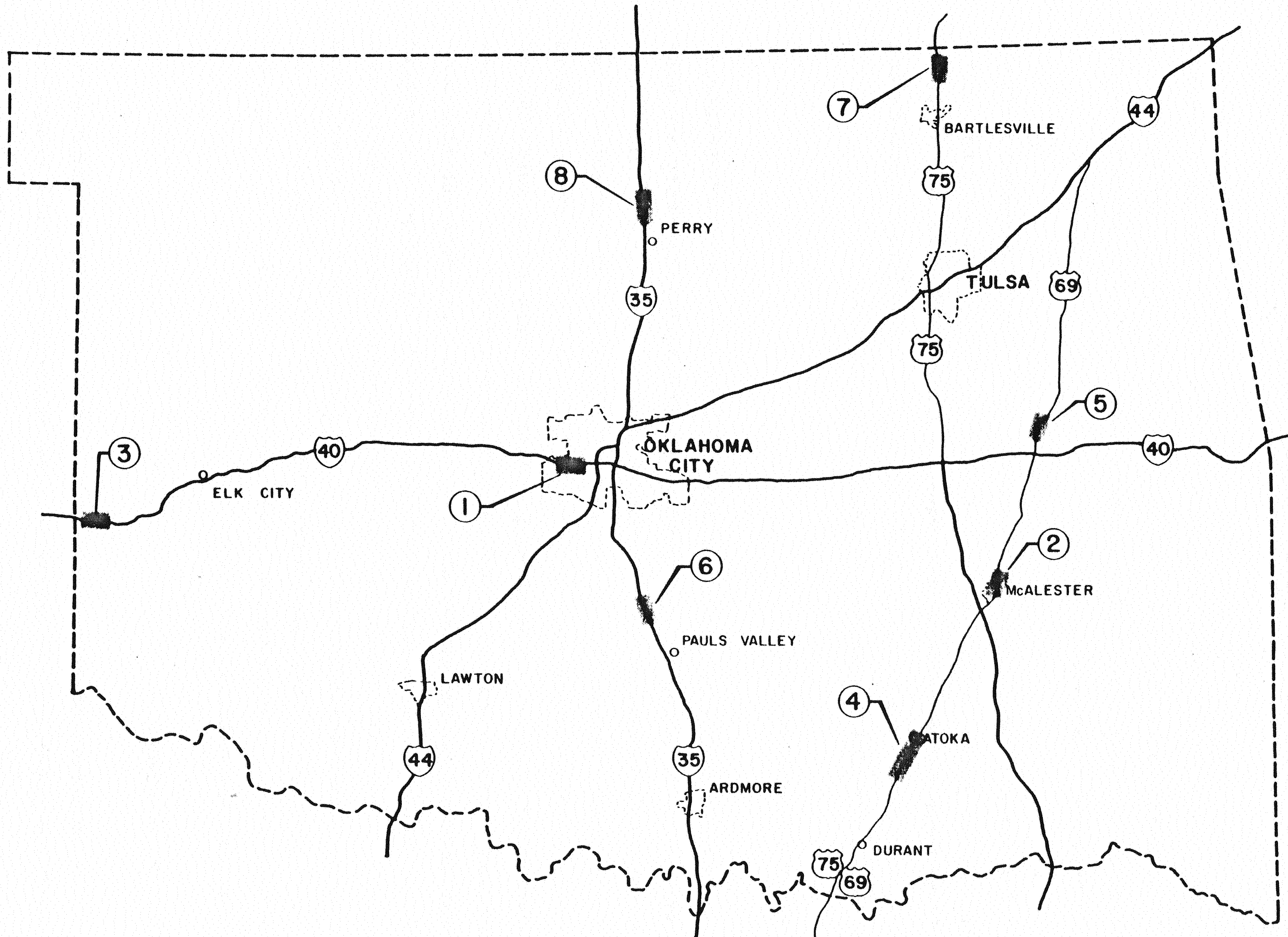

Figure 1. Locations of eight study sites in Oklahoma.

OBJECTIVES OF RESEARCH PROJECT

The objective of this research project was to investigate several early pavement failures, review Oklahoma's pavement design procedures and management practices, and make recommendation for revised pavement design procedures and management practices.

The Phase I report covers the details of the eight pavement sections selected for study to accomplish these objectives. It covers the tests performed and recommendation for improvement of ODOT material selection practices, reasons for early failures of the 6 sections selected, and possible corrective treatments for each of these sites.

SCOPE OF RESEARCH PROJECT

Phase-I of this project consisted of interviews of ODOT officials, diagnostic evaluation by an expert team, and both laboratory and non-destructive field evaluation of surfaces, bases and subgrades. From these investigations, reasons for premature failure of the six pavement sites were determined and reported to ODOT (3). The second phase of this project was to review the Oklahoma pavement thickness design guide (4) and management practices and to compare these practices with the revised AASHTO pavement design guide (2). The third and final phase of this work was to make recommendations as necessary to improve ODOT design procedures, material specifications, and pavement management activities. The above recommendations are based on the results of the first two phases of the project. They are submitted to ODOT for possible use in modifications of the currently practiced pavement design and management procedures, and material specifications.

OBJECTIVE OF REPORT

The objective of this report is to submit necessary recommendations to ODOT for upgrading the currently used pavement design and management practices and material specifications. These recommendations are based on a detailed review of current pavement design and management practices and the study conducted in Phase-I (3) of this project.

SCOPE OF REPORT

Research findings of Phase II and III are documented herein. A detailed review of the ODOT pavement design and management procedures is described in Chapter 4, as well as a comparison of ODOT and AASHTO pavement design procedures. Chapter 5 provides recommendations for improvement of the existing practices and/or specifications.

CHAPTER 2
INTERPRETATION OF OBSERVATIONS

In the preliminary report (3), substantial data were presented relative to performance, laboratory measurements, etc. In order to condense the information into a manageable and understandable level, a factorial analysis was used. Generally, one determines the observations based on a factorial, but in this case, a factorial was developed to fit the collected data.

A factorial of two levels was used for each variable since only a limited number of sections were available for analysis. Using these factorials and the process of elimination, only significant variables were identified. This factorial analysis also helped identify the significant data. The most fruitful thrust of the analysis is summarized in the following sections: determining the significant distress types of rutting followed by a discussion of the stripping mechanisms. The last section explains a method to identify the stripping potential of an aggregate.

PRIMARY DISTRESS CONSIDERATION

To study the relationships between the observed primary distress and pavement performance, a factorial analysis was conducted using rutting and cracking of the pavement as independent variables. A pictorial representation of this analysis is shown in Figure 2. In this analysis, only Sites 1 and 7 are assumed to have good performance records. ODOT staff engineers made these subjective judgments about the sites in outlining the project scope. The Present Serviceability Rating (PSR) value for each pavement site is also shown in Figure 2 inside the triangular symbol. These values of PSR along with several other data for the factorial analysis are shown in Table 2. The determination of high and low rutting and cracking is based on the results of condition surveys described in the Phase-I report (3). Figure 2 illustrates that the two

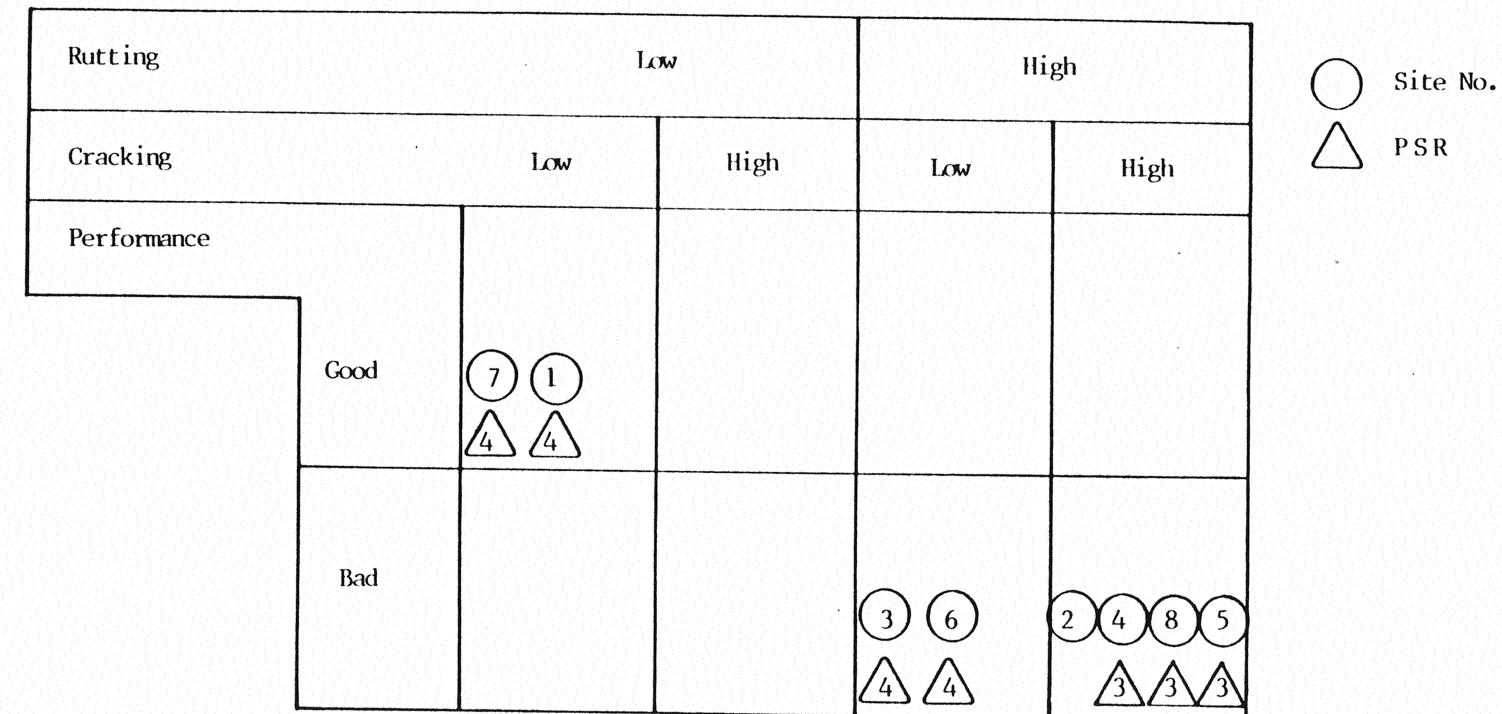

Figure 2. Factorial analysis of rutting, cracking, and pavement performance.

Table 2. Data for factorial analysis

Site No.	PSR[1]	Asphalt[2] Retained (%)	P_{RS}[3] (%)	Avg.[4] Void (%)	Traffic[5]
1	4	--	--	--	11.60
2	--	74	62	10	1.30
3	4	56	--	13	4.50
4	3	78	19	7	1.90
5	3	80	65	13	3.10
6	4	57	39	--	9.40
7	4	82	27	5	0.45
8	3	45	88	5	2.15

(1) PSR = Present Serviceability Rating

(2) Lottman Test

(3) P_{RS} = Percent Retained Strength

(4) Lottman Test

(5) Estimated number of 18 kip EAL since construction in millions

good performing sites are located in the areas of low cracking and low rutting. All the bad performing sites are located in the areas of high rutting having either low or high cracking. Thus it was hypothesized that both rutting and cracking are significant variables in defining bad pavement performance. Good performance was indicated by low cracking and low rutting.

In contrast to cracking, Figure 2 shows that pavement sites of satisfactory performance are located in the areas of low or no rutting, and those showing poor performance are associated with a large amount of rutting. Thus, there appears to be an excellent correlation between rutting and pavement performance as defined by ODOT. Distortion could be due to a shearing failure either in the roadbed level or in the upper layers as a result of inadequate base materials or overstressing of the roadbed materials. Since the base layers are asphalt stabilized materials, asphalt stripping reduced the strength and led to shearing fractures and consequently rutting. In the next section stripping is considered.

ANALYSIS OF STRIPPING

For the stripping phenomenon to occur in a pavement structure, water, heavy traffic, and moisture susceptible aggregates must be present. A factorial analysis of the two levels for each of the above mentioned variables was performed to study their contribution to stripping. This factorial analysis also helped in understanding the more damaging combinations of these variables in the stripping of aggregates. This analysis is shown in Figure 3.

The following was used to assign criteria for each of the test sites in Figure 3 in order to separate good and poor performance.

a) If asphalt retained in the Texas Boiling Test is less than 70 percent, it may be considered as a potential stripping material.

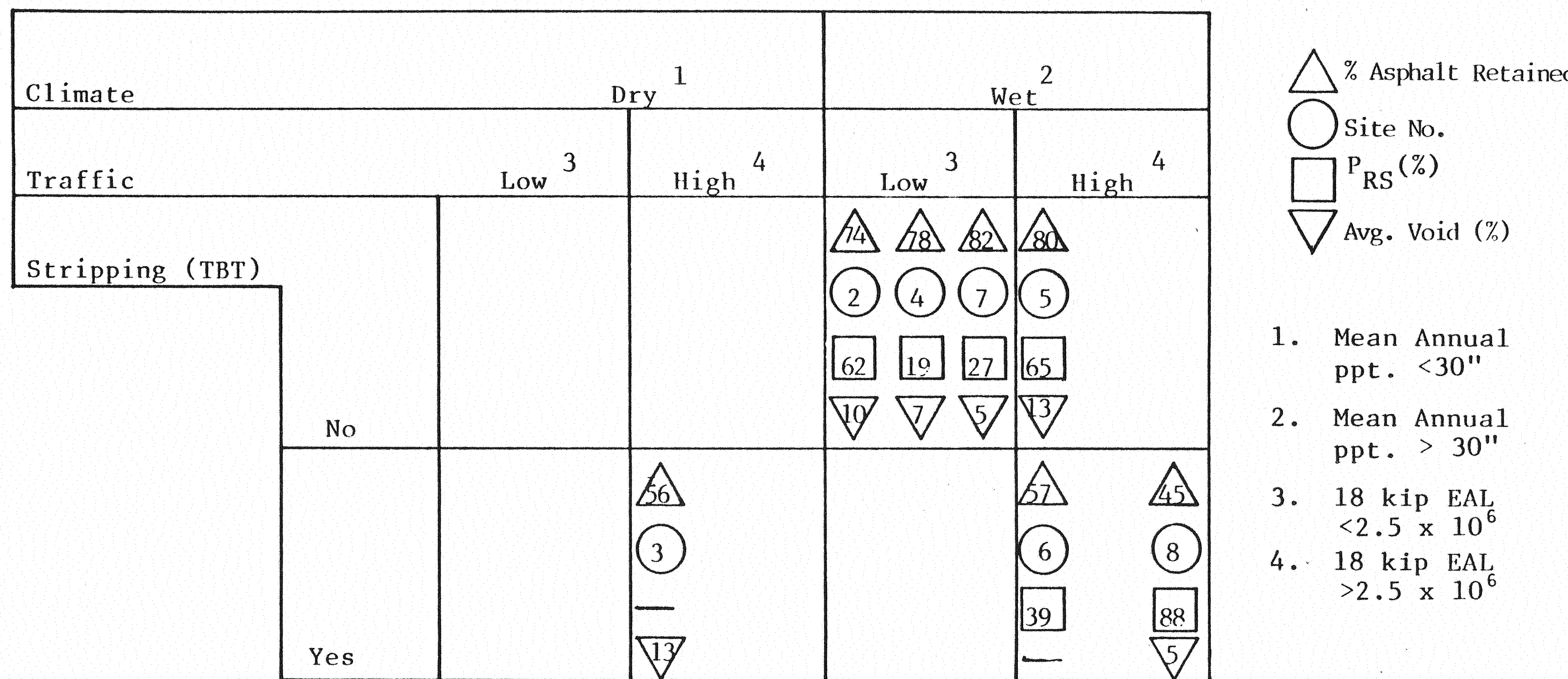

Figure 3. Factorial analysis of climate, traffic, and stripping.

b) Cumulative traffic of 2.5 million 18 kip EAL's since construction was used as the dividing line between high and low traffic.

c) A mean annual precipitation of 30 inches was used as the separation line for a dry and wet climate. A mean annual precipitation map showing the isolines for the state of Oklahoma (5) is shown in Figure 4. These isolines are based on the data for the period of 1931 to 1955.

Figure 3 shows no definite relationship indicating the effect of traffic and climate on the aggregate stripping in the pavement structures. This figure also failed to show any significant effect of percent air voids on stripping. This observation leads to the conclusion that, although stripping should be more significant for higher values of traffic, moisture, voids, or any combinations thereof, the stripping occurring in the project sites is probably due primarily to the presence of moisture susceptible aggregates.

Figure 3 shows the values of percent asphalt retained from the Texas Boiling Test, average void ratio, and percent strength retained (P_{RS}). These values are also shown in Table 2. The values of P_{RS} were computed by using the tensile strengths of the specimen from the Lottman test and Eq. 1.

$$P_{RS} = \frac{\begin{bmatrix}\text{Dry} \\ \text{Strength}\end{bmatrix} - \begin{bmatrix}\text{Conditioned} \\ \text{Strength}\end{bmatrix}}{\begin{bmatrix}\text{Dry} \\ \text{Strength}\end{bmatrix}} \qquad (1)$$

Similar analysis was conducted using the Lottman test results to determine the stripping potential of an aggregate used in the study sites. This factorial analysis is shown in Figure 5. This analysis indicates a definite relationship between the climatic conditions and the stripping of aggregates in the pavement structure. Using the criteria defined above, six of the seven sites investigated showed significant stripping. All of these are located in the wet region. The only site that did not show a

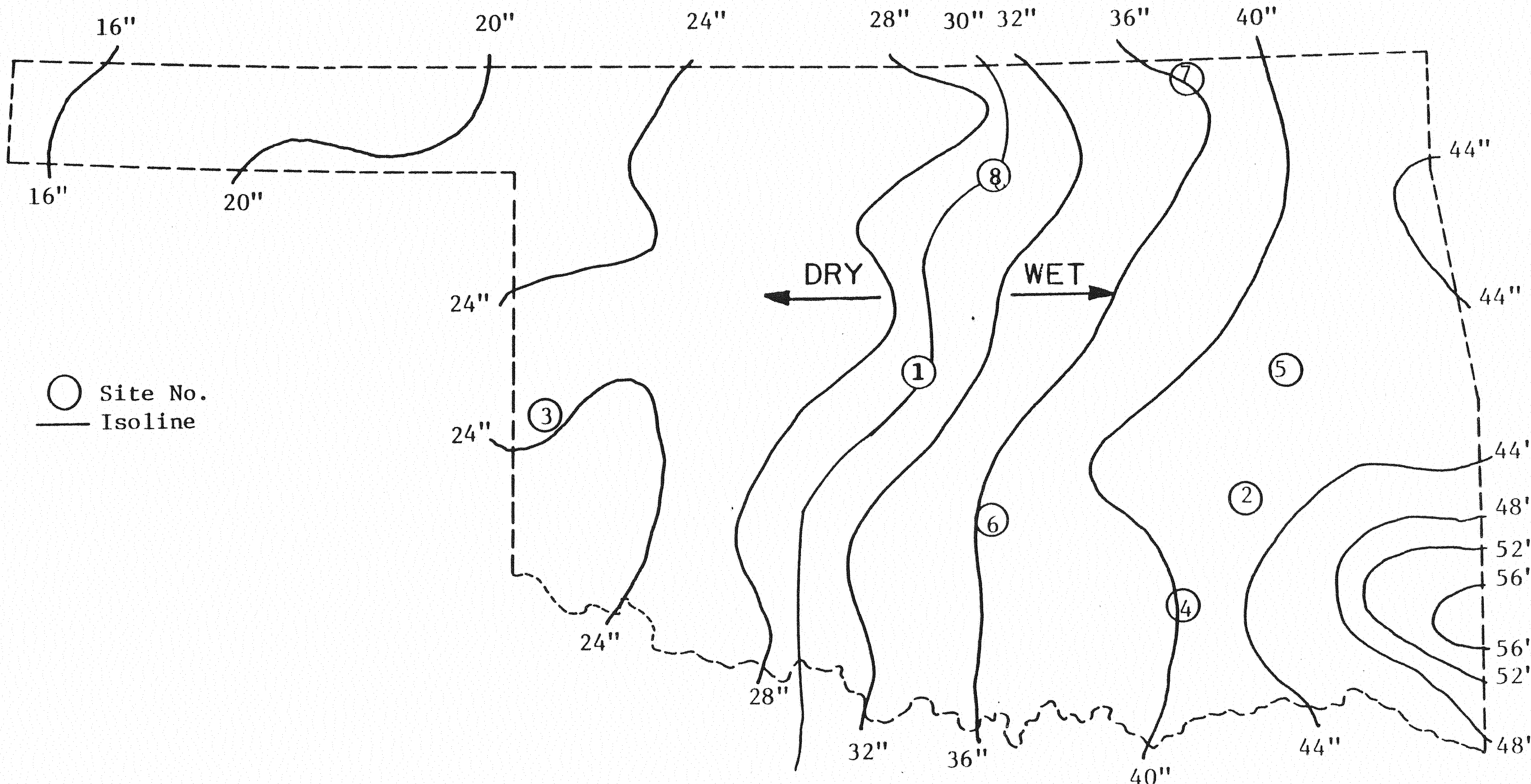

Figure 4. Mean annual precipitation in inches for Oklahoma with site locations.

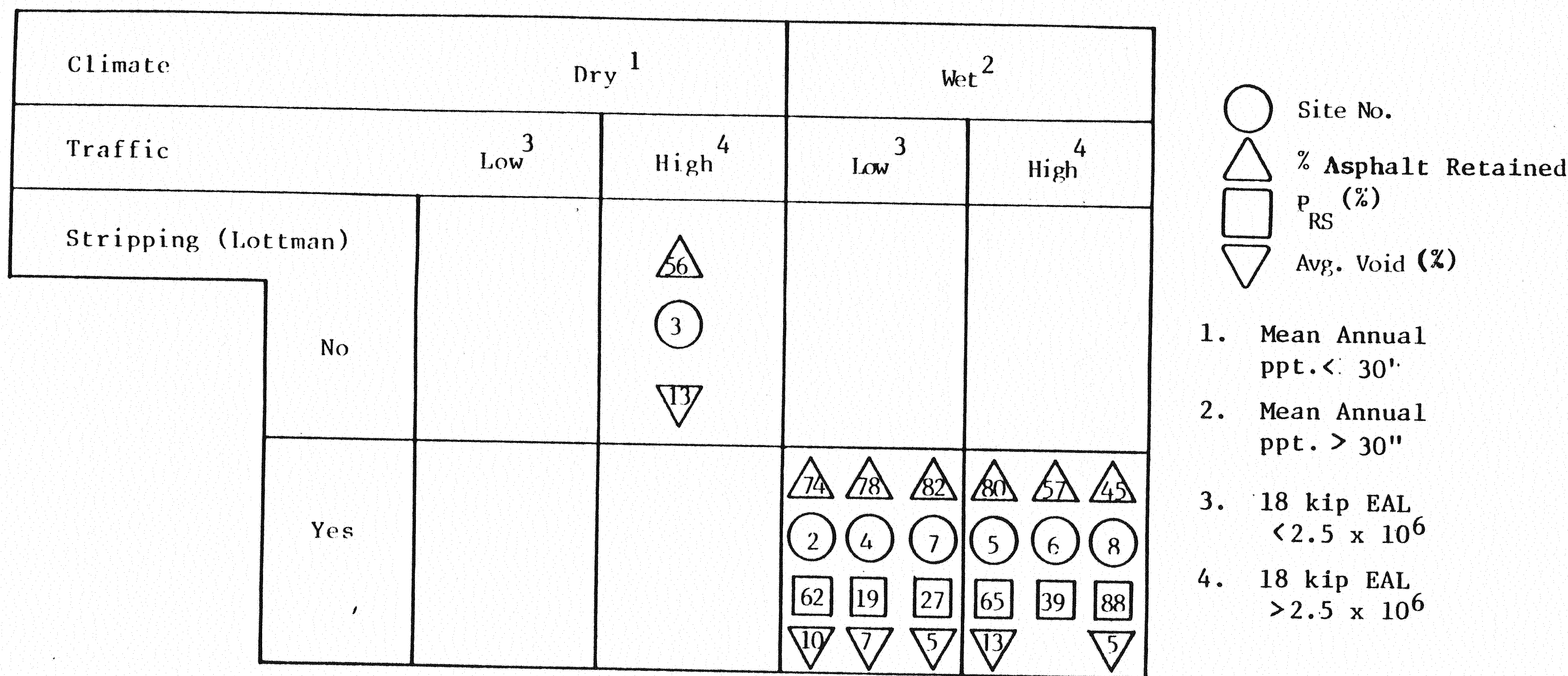

Figure 5. Factorial anaysis of climate, traffic and stripping.

significant amount of stripping is Site No. 3 which is located in the dry climate area. This postulates that stripping is occurring due to the presence of moisture, and the Lottman test (1) may be used to identify stripping.

TEST METHOD TO EVALUATE FIELD PERFORMANCE

The effect of climatic conditions and traffic volumes on pavement performance was studied by a factorial analysis method using the previously defined criteria to distinguish between dry and wet conditions of climate along with high and low traffic volumes. Results of this factorial analysis are shown in Figure 6. In this analysis only sites 1 and 7 are assumed to have good performance records. A definite relationship between pavement performance and climatic condition is evident. Five out of six pavement sites indicating unsatisfactory performance are located in the wet region of the state having an average annual rainfall in excess of 30 inches. These five sites include both rigid and flexible pavements. The remaining site No. 3 indicating poor performance and located in the dry climate region carries a large volume of traffic. One of the two pavement sites with a good performance record is located in the wet region but the traffic volume is small. The other good site is also located in a wet climate area and carries a large volume of traffic.

SUMMARY

The observations from the factorial analysis of climate, traffic, and pavement performance indicate that, with the existing material specifications, the pavements performed poorly if the climate was wet or the traffic volume was high. A combination of these two factors caused even more pavement deterioration. Furthermore, the Lottman Test provides a method of indentifying the potential of aggregate stripping.

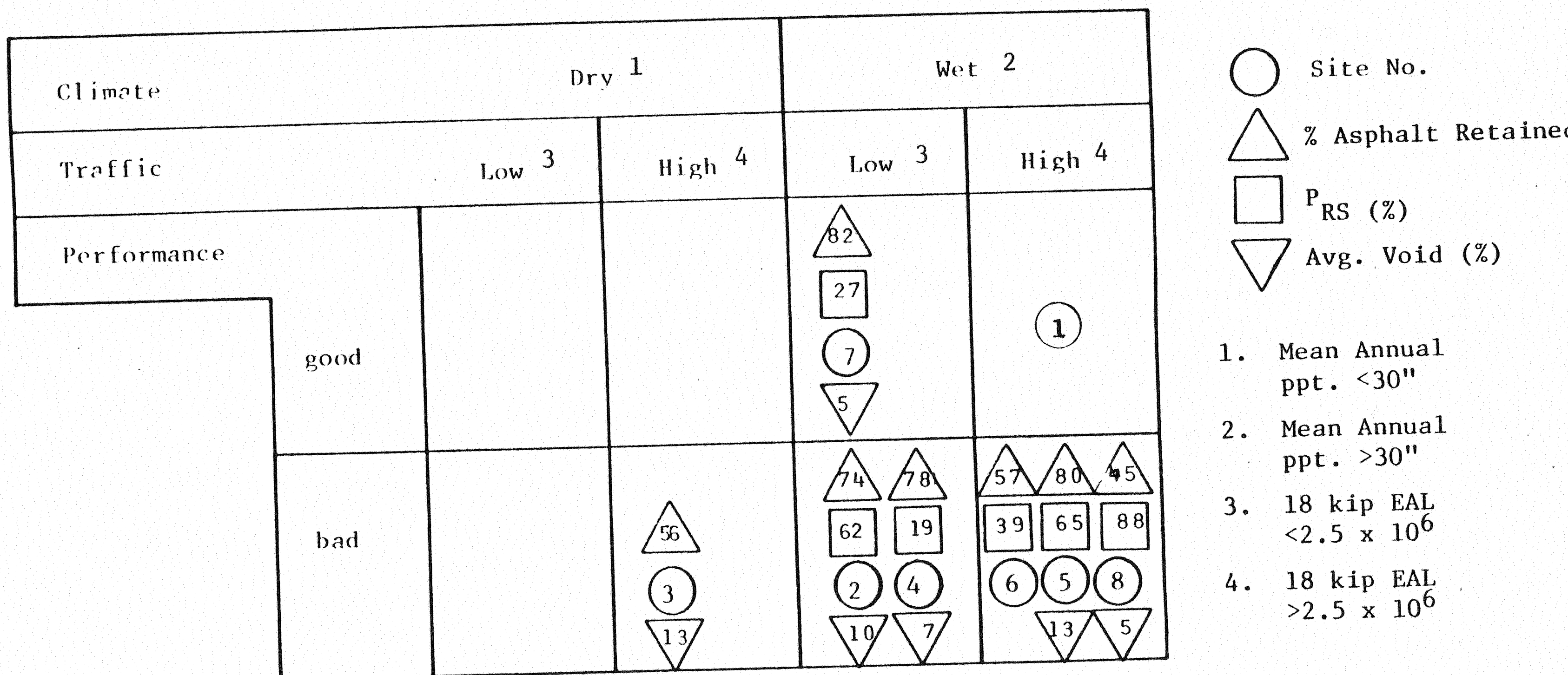

Figure 6. Factorial analysis of climate, traffic, and pavement performance.

The criteria separating good and poor performance; i.e., leading to stripping, are defined in the section on the analysis of stripping. It is also emphasized that these results are based on a limited number of test sections. Thus, a more detailed analysis should be developed to establish criteria identifying a potential stripping condition; i.e., poor performance. For example, one criteria may be developed as follows: a material with Lottman values of less than 70 should not be used with an annual rainfall greater than 30 inches and a design traffic greater than 2.5 million vehicles during the design life. Only by observing numerous pavements over the state could realistic design and specification criteria be developed.

CHAPTER 3
DEFLECTION PROCEDURE

The primary purpose for measuring the deflection of an existing pavement is to determine if the structural strength is adequate to carry the existing or predicted traffic for the design service period. The revised AASHTO guide (2) for design of pavement structures stresses the importance of determining the modulus properties of the subgrade by use of deflection measurements. The revised guide uses resilient modulus for characterizing the material properties for determining the pavement structure thickness required for a given subgrade. In this analysis a computer program entitled OVERLAY was used to backcalculate the pavement structure properties; i.e., resilient modulus of the layers based on deflection. This chapter presents a comparison of various deflection measurement devices used in this study to ascertain if one gives a better prediction of the properties.

NDT DEVICES

Three instruments commonly used for deflection measurements for the evaluation of pavements and subgrades are the Benkelman Beam, Dynaflect, and Falling Weight Deflectometer (FWD). A listing of advantages and disadvantages of various NDT devices are shown in Table 3. The Benkelman Beam data was collected by ODOT for the test sites but was not analyzed for this phase of the study since a deflection basin is required to backcalculate structural properties of materials. Moduli and other pertinent properties were determined for all the project sites using both Dynaflect and FWD data. A detailed description of the data collection and evaluation procedures for these devices are given in the Phase-I report (3) of this study. This nondestructive deflection testing and analysis included the eight sites, covering rigid, flexible, and composite pavements.

Table 3. Advantages and disadvantages of NDT devices

Device	Advantages	Disadvantages
Benkelman Beam	(1) Simple to set up and operate.	(1) Slower operation compared to dynamic devices.
	(2) Maximum surface deflection is measured under the design wheel loading.	(2) Requires a loaded truck to measure under rebound deflection.
	(3) Availability of data bases for empirically predicting load carrying capacity.	(3) Does not necessarily simulate the dynamic loading at normal highway speed.
	(4) Benkelman Beam itself costs less than $1000.	(4) A deflection basin is not measured.
		(5) The deviation of insitu elastic moduli and application of mechanistic analysis is not possible for rehabilitation design.
		(6) Operating costs are higher than Dynaflect or FWD.
Dynaflect	(1) A light load automated device that applies harmonic loading.	(1) The loading mode does not simulate exactly the signal of a moving vehicle.
	(2) Equipment set up, operation, and geophones calibration are simple and fast.	
	(3) Deflections measured by five geophones are used to form a deflection basin.	

Table 3. Advantages and disadvantages of NDT devices (Contd.)

Device	Advantages	Disadvantages
Dynaflect (contd.)	(4) Reliable in operation and accurate in measurements. (5) Most widely accepted dynamic load device among state highway agencies [a]. (6) Several computer programs [b] are available for mechanistic interpretation of deflection basins in order to determine in situ moduli and load carrying capacity. (7) Correlations with Benkelman Beam are available in literature [c]. Several agencies have over 15 years experience with Dynaflect testing. (8) The capital cost is in the order of $20,000 (inexpensive compared to FWD). (9) Operating and maintenance cost are lower than other NDT devices.	

Table 3. Advantages and disadvantages of NDT devices (Contd.)

Device	Advantages	Disadvantages
FWD	(1) A dynamic load device capable of applying a range of transient loads.	(1) The transient signal of FWD has the same loading mode as a fast moving vehicle but the shapes of the two signals are not similar. Duration of transient signal under a fast moving wheel is several times larger than the FWD load signal.
	(2) Recent models are easy to operate and fully automated.	(2) Calibration of geophones is not fully possible on routine basis.
	(3) Deflections measured by an array of geophones are used to form a deflection basin.	(3) It costs several times more than a Dynaflect. (e.g., the Dynatest FWD mode 8000 costs arount $85,000).
	(4) Testing is fast compared to the Dynaflect testing at preset load level.	(4) Recent models are being evaluated and used by several agencies. Performance data for long term operation and maintenance are not available.
	(5) Deflection basins can be analyzed using elastic layered theory to determine insitu elastic moduli and load carrying capacity [b].	(5) Requires careful evaluation for selecting a system from several available models.

a. Report No. FHWA/RD-83/097, "Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements," by R.E. Smith and R.L. Lytton, April 1984.

b. "Project-Level Structural Evaluation of Pavements Based on Dynamic Deflections," by W. Uddin, A.H. Meyer, W.R. Hudson and K.H. Stokoe. Transportation Research Record No. 1007, 1985.

c. MS-17, The Asphalt Institute, June 1983.

Since the devices use different pavement loadings, a comparison of the deflection magnitude is irrelevant. The primary question is; "Do the measurements yield the same layer properties when used in the OVERLAY computer program?" Thus, the precept of the study reported in the following section is that if the properties are similar, the deflection devices are equivalent.

COMPARISON OF NDT DEVICES

A computer program was used to determine the moduli properties for each individual layer of the pavement structures. Deflection measurements from both the Dynaflect and FWD were used as input data to determine the moduli properties. This section presents a comparison between the data obtained from both instruments.

Site Nos. 2 and 5 were chosen as representative of flexible and rigid pavements respectively to show the relationships between the moduli obtained from both the Dynaflect and FWD. The moduli values are compared in Figure 7 for Site No. 2. The regression line describing the relationship between moduli obtained by the FWD and Dynaflect is given by

$$E_{FWD} = 28.44\,(E_{Dynaflect})^{0.704} \qquad (2)$$

the power function of Eq. 2, having a correlation coefficient, R=0.866. This figure also shows a line of equality for comparing the moduli values. Although, this figure shows a large scatter of data points and has a low correlation coefficient, the number of points above and below the line of equality are evenly distributed. This indicates that there is a relationship between the two sets of moduli values for this flexible pavement site. The small value of the correlation coefficient is partly due to the large scatter of data points.

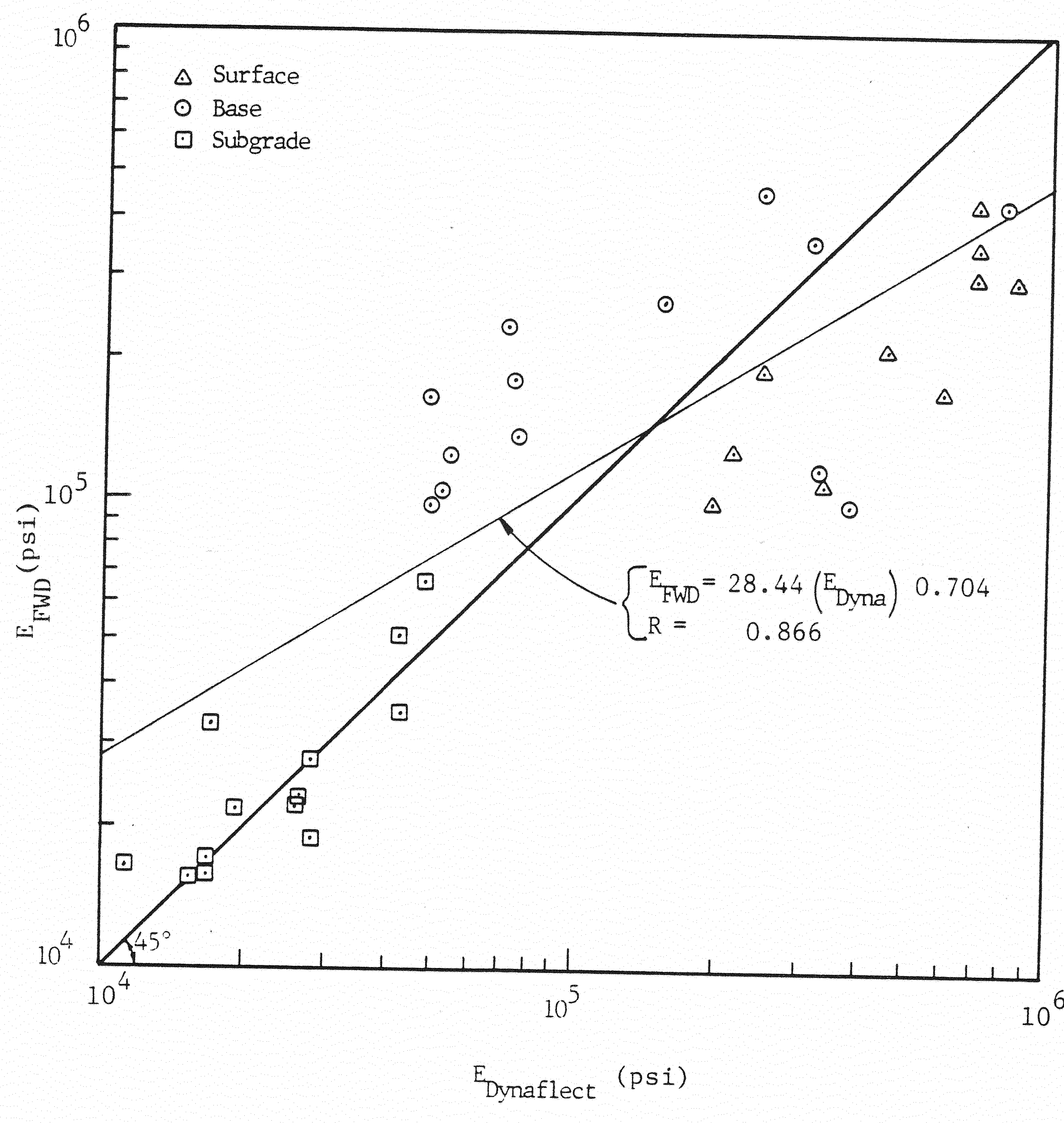

Figure 7. Typical relationships betweem E_{FWD} and $E_{Dynaflect}$ for flexible pavement at northbound lanes of Site No. 2.

The comparison of moduli values for Site No. 5 is shown in Figure 8. The regression line describing the relationships between the moduli values obtained from the FWD and Dynaflect measurements are given by Eq. 3.

$$E_{FWD} = 0.794\ (E_{Dynaflect})^{1.006} \tag{3}$$

The correlation coefficient for this power function is high and has a value of 0.974. The line of equality of Figure 8 and the high value of the correlation coefficient indicate there is a definite relationship between the two moduli. Stronger relationship between the two moduli is due to smaller scatter of the data.

The standard deviations of the moduli values for the Dynaflect and FWD were computed for each site. The standard deviations of the difference between the moduli values were also computed. These values are shown in Table 4. This table shows that standard deviations of moduli obtained by the Dynaflect and FWD are much higher than the standard deviations of their differences. On the basis of this observation, it can be concluded that definite relationships exist between the moduli obtained by using Dynaflect and FWD even with the large scatter of data points as shown in Figure 7 and 8.

The relationships between the average values of E_{FWD} and $E_{Dynaflect}$ for rigid and flexible pavement sites were investigated. Plots were made using the average moduli values for the surface, base, subbase, and subgrade. These plots for rigid pavement sites are shown in Figure 9 and for flexible pavements in Figure 10. In these figures moduli were averaged to obtain mean values but direction of traffic was treated independently. For example, average moduli for the northbound lanes were treated independently from that of south bound lanes. The power functions describing the regression lines for the plots of rigid and flexible pavements are given by Eqs. 4 and 5 respectively.

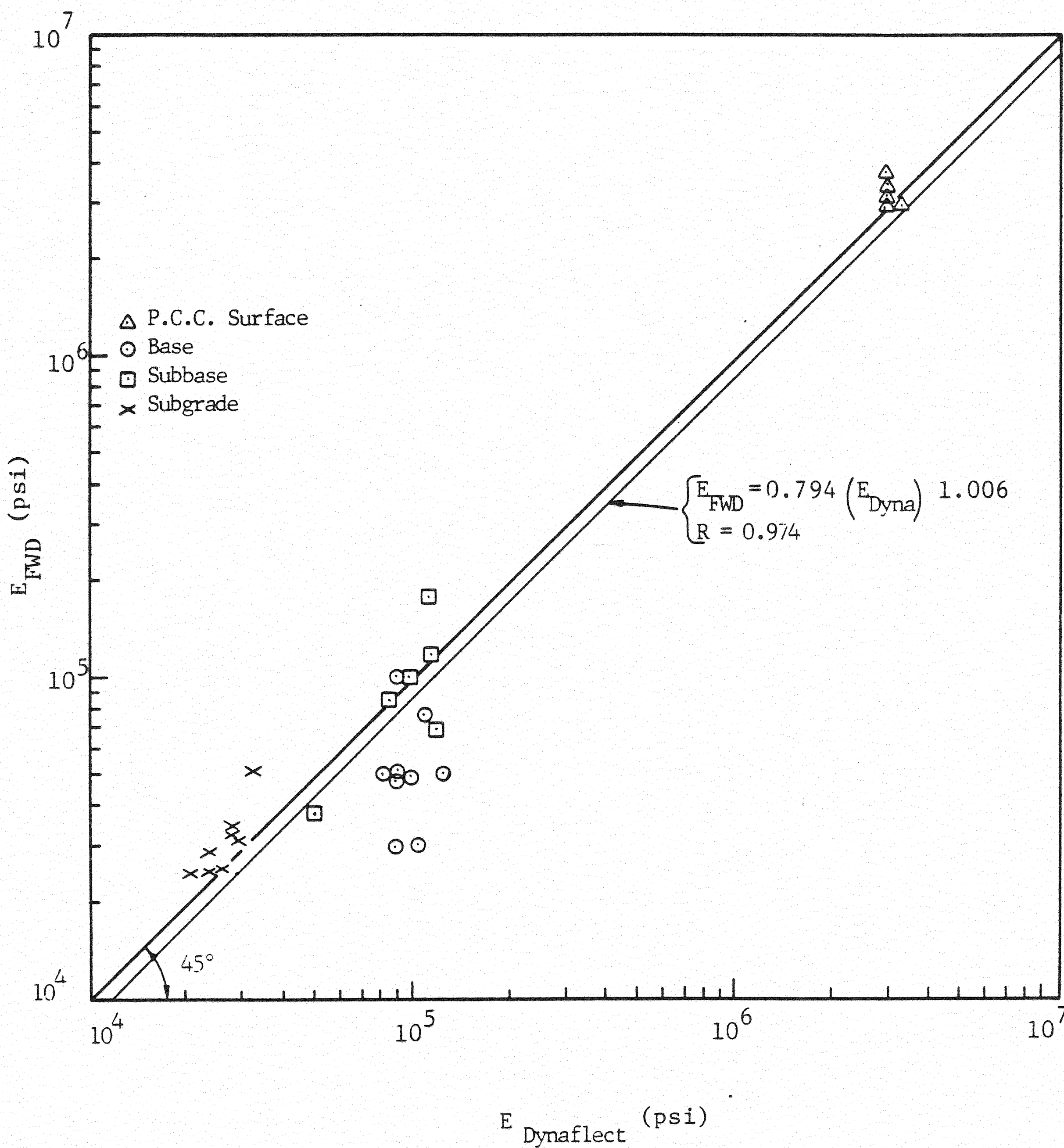

Figure 8 Typical relationships between E_{FWD} and $E_{Dynaflect}$ for rigid pavement of Site No. 5.

Table 4. Standard deviations of moduli for site Nos. 2 and 5

Pavement Type	Site No.	Standard Deviation		
		E_{FWD}	$E_{Dynaflect}$	$(E_{FWD} - E_{Dyna})$
Rigid	2	133,000	229,000	184,000
Flexible	5	1320,000	835,000	185,000

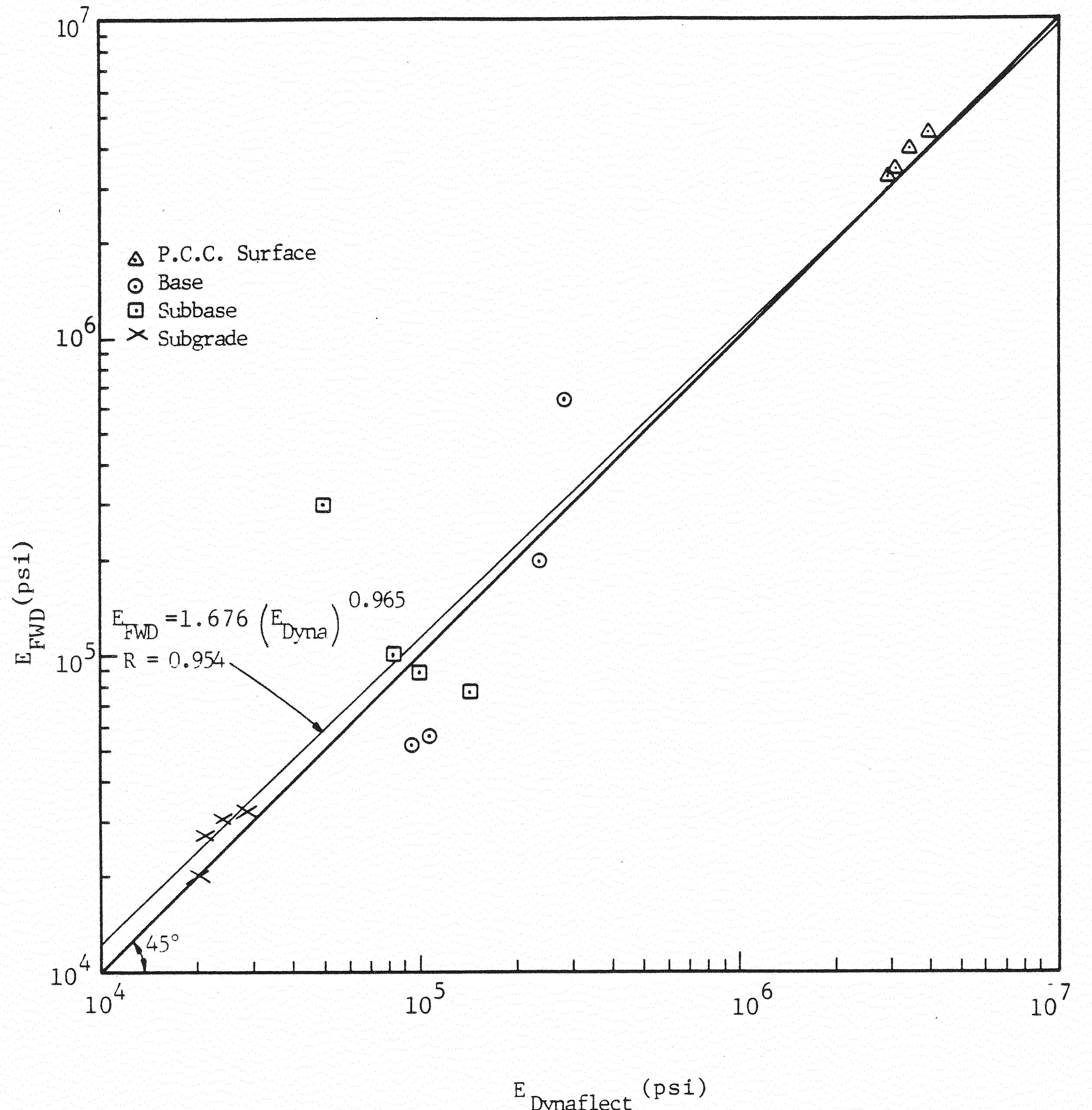

Figure 9. Relationships between average values of E_{FWD} and $E_{Dynaflect}$ for rigid pavement sites.

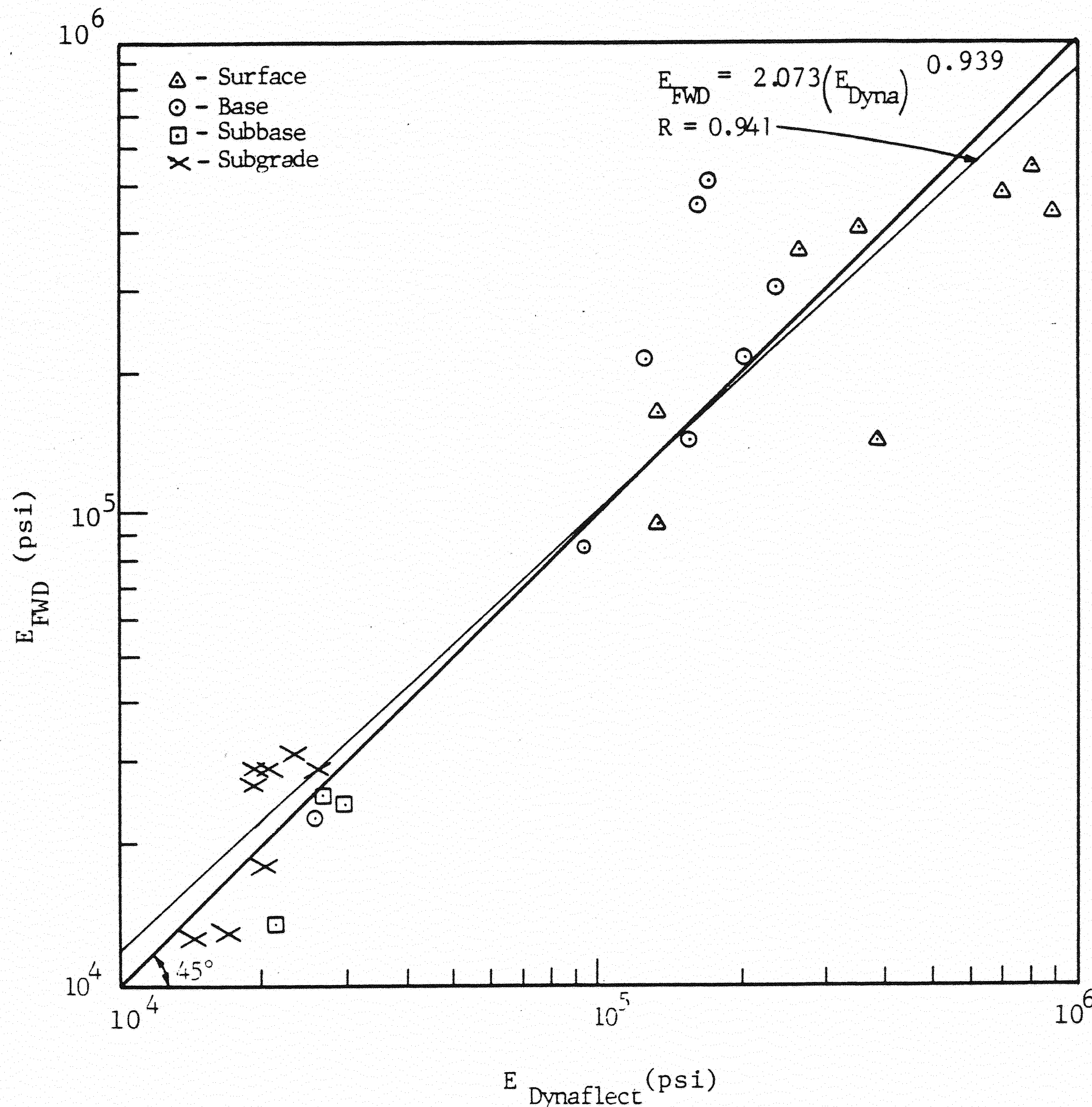

Figure 10. Relationships between average values of E_{FWD} and $E_{Dynaflect}$ for flexible pavement sites.

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$$E_{FWD} = 1.676\ (E_{Dynaflect})^{0.965} \tag{4}$$

$$E_{FWD} = 2.073\ (E_{Dynaflect})^{0.939} \tag{5}$$

The correlation coefficient of Eq. 4 is 0.954 and that of Eq. 5 is 0.941. These high values of the correlation coefficients and low scatter of data points around the line of equality show that the average values of the E_{FWD} and $E_{Dynaflect}$ are closely related. This indicates that irrespective of the equipment used for deflection measurements, the average value of moduli obtained for various layers of pavement represents the structural strength. So either the Dynaflect or FWD can be used to determine moduli values used in pavement design.

CHAPTER 4

REVIEW OF ODOT PAVEMENT DESIGN AND MANAGEMENT PROCEDURE

The ODOT pavement design and management procedure was reviewed. A sensitivity analysis was performed to study the importance of climate, shoulder, and traffic factors in determining the pavement thickness. Designs of the eight pavement sites were checked to identify any discrepancy between the manual and designs.

ODOT PAVEMENT DESIGN PROCEDURE

The thickness design procedure (4) currently practiced by the ODOT was first implemented in 1962 and revised in 1965. Basically, there are two different steps involved: (a) design the pavement to match the existing subgrade; and (b) design the subgrade to match the predetermined pavement. The first is a preliminary design, based on the pedological soils and underlying geology. Secondly, a final design is prepared based on the top two feet of finished subgrade, following the grading operation.

Both the preliminary and final design are based on the Oklahoma Subgrade Index (OSI). The OSI value is determined from the known properties of subgrade; namely, liquid limit, plasticity index, and percent passing the No. 200 sieve (4) as shown in Figure 11. The design procedure directly uses the OSI values and the wheel load to determine required Equivalent Base Thickness (EBT) in inches (4) as shown in Figure 12. The next step involves adjusting the EBT using the shoulder, traffic, and climatic factors. The final layer thickness is then determined from the adjusted EBT using material equivalence factors. A flow chart of the steps involved in determining pavement thickness is shown in Figure 13.

The procedure to calculate the final thickness is based on the relationship given in Eq. 6 suggested by AASHTO (2). In this equation a_1,

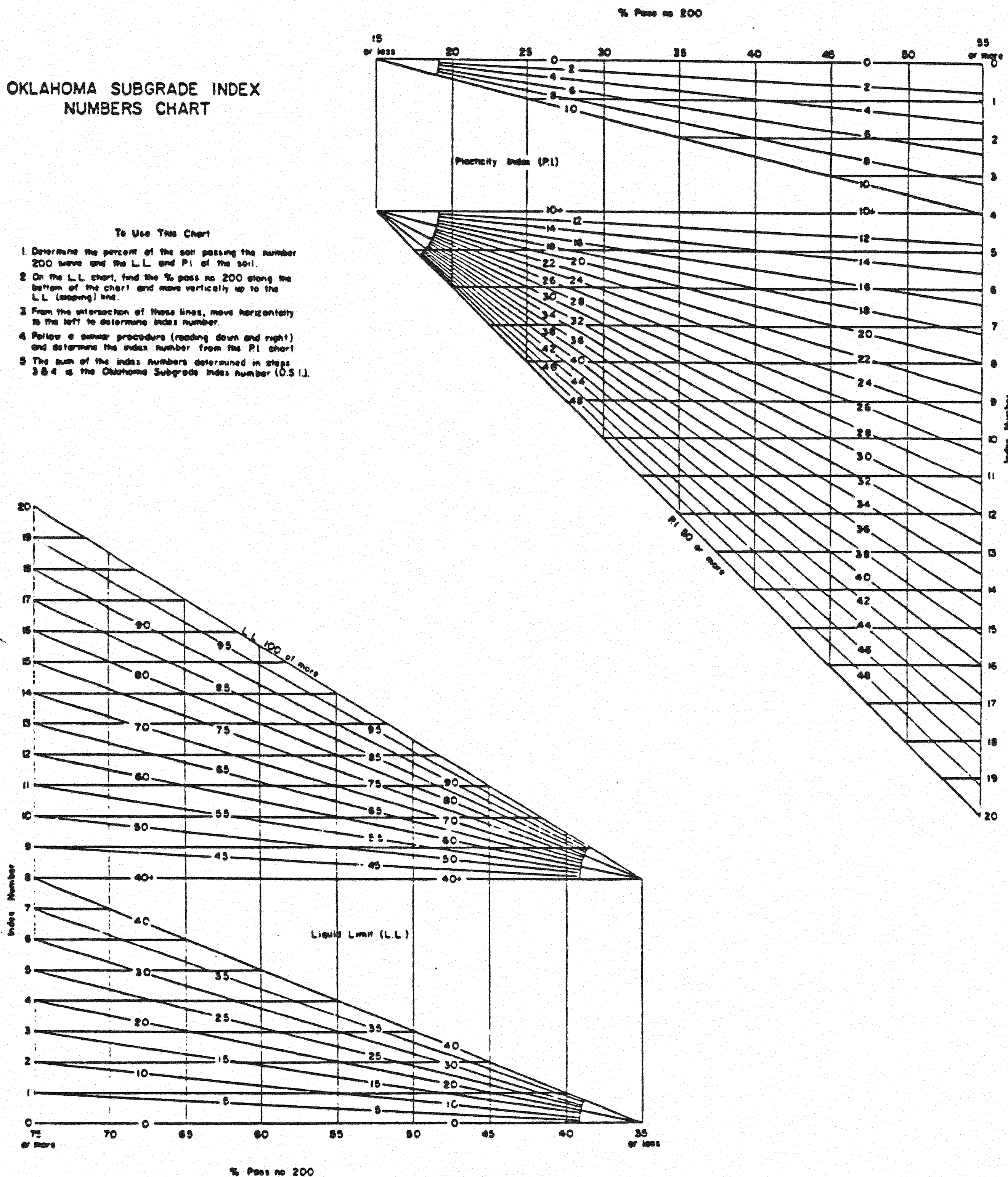

Figure 11. Determination of OSI.

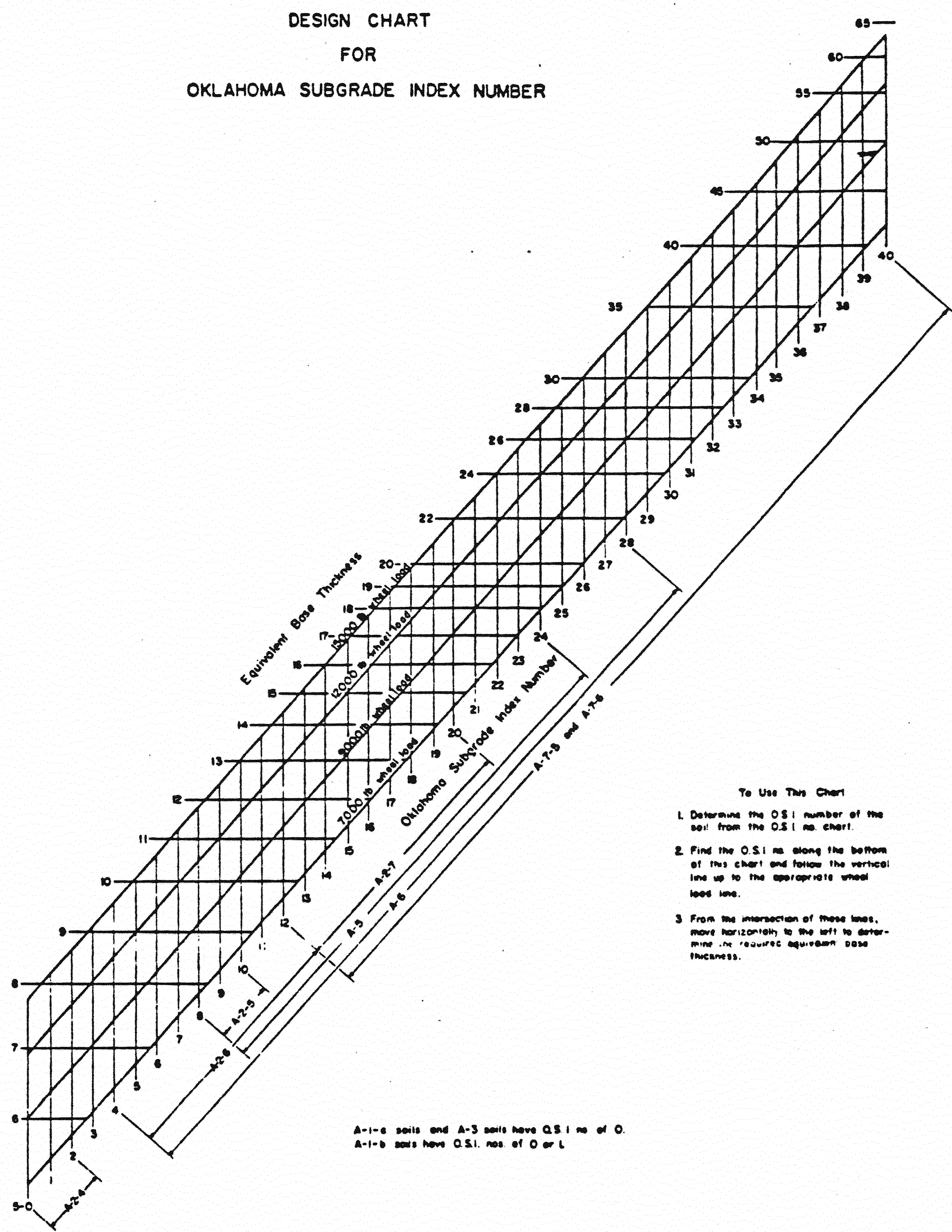

Figure 12. Determination EBT.

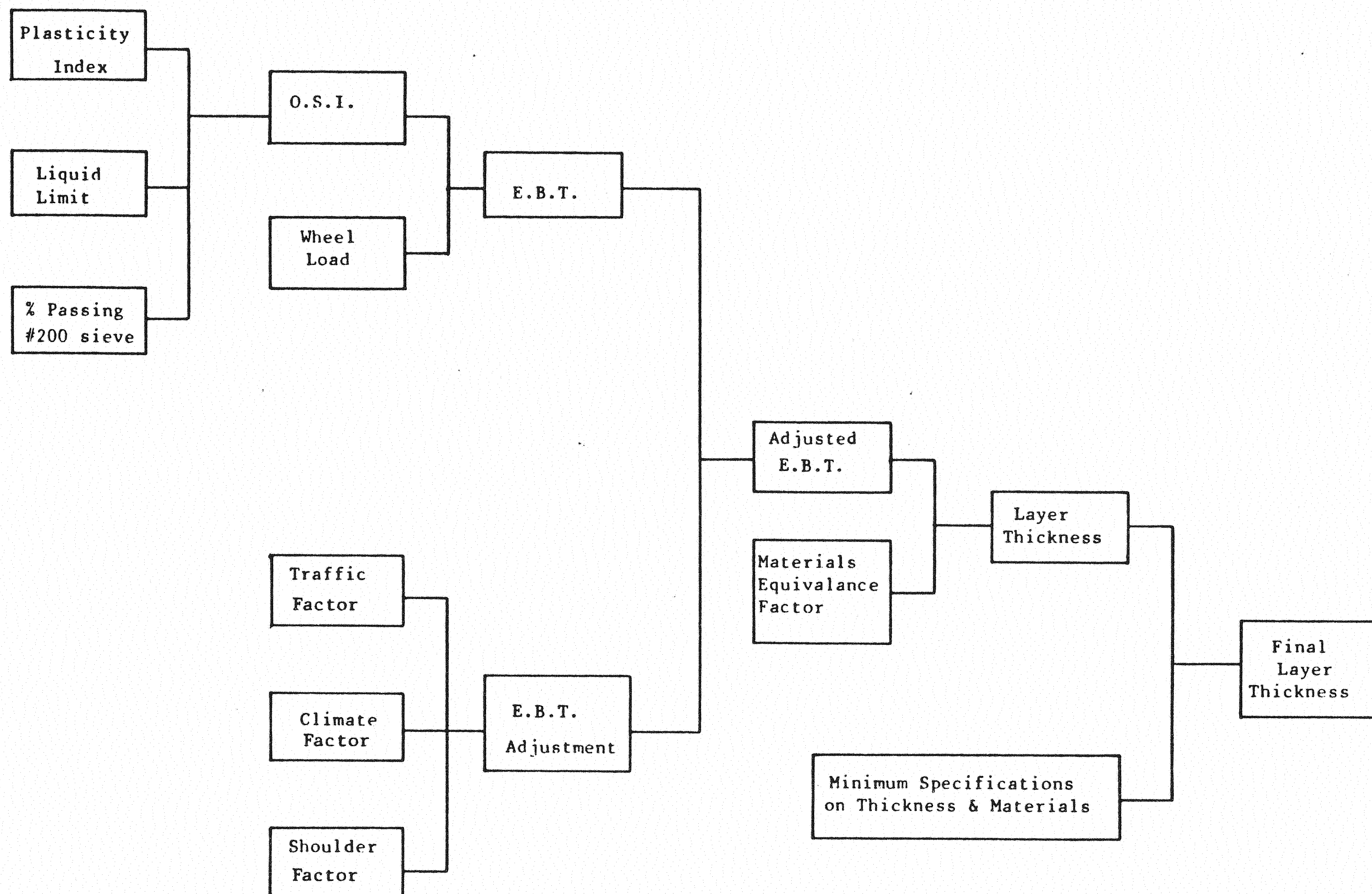

Figure 13. Flow chart for ODOT pavement thickness design procedure to match the existing subgrade.

a_2, and a_3 are the layer equivalence factors for surface, base, and subbase respectively and d_1, d_2 and d_3 are the corresponding thicknesses.

$$EBT = a_1d_1 + a_2d_2 + a_3d_3 \tag{6}$$

In addition to the factors considered to determine the EBT, special adjustments must be made to account for subgrades having a plasticity index of 25 or more, and the potential vertical rise (PVR) of the soil.

SENSITIVITY ANALYSIS OF PAVEMENT THICKNESS ADJUSTMENT

Adjustments of pavement thicknesses for shoulder considerations are based on the material, surfacing and width considered by the designer. The rating of the shoulder factor may range from 0 to 20 (4). These ratings are shown in Table 5. The climate factor ranges from 15 to 40 (4) and is shown in Figure 14. The traffic factor is the number of overloaded axles per day and is defined by Eq. 7.

$$\begin{pmatrix}\text{Traffic}\\ \text{Factor}\end{pmatrix} = \text{ADT}\begin{pmatrix}\text{Percent Heavy}\\ \text{Commercial Trucks}\end{pmatrix} \times \begin{pmatrix}\text{Percent Commercial}\\ \text{Overload Axles}\end{pmatrix}$$

Adjustments to the design EBT are determined from a nomograph using the three factors; shoulder, traffic and climate, as shown in Figure 15 and Table 6.

To study the sensitivity of the three contributing factors, namely traffic, climate, and shoulders, three values for each factor were chosen to represent the entire spectrum. The adjustment thickness obtained for various combinations of values of the aforementioned factors using the Oklahoma Pavement Design Guide (4) are shown in Table 7. This table shows the sensitivity of thickness adjustments on the three above mentioned factors.

Table 5. Shoulder Factors

Material	Surfacing	Width(ft.)	Rating
Suitable soil	None	1-2	0
Suitable soil	4" gravel	1-2	1
Soil Asphalt	None or single bit.	1-2	2
Stabilized aggregate	None	1-2	2
Stabilized aggregate	Single bit.	1-2	3
Suitable soil	None	3-5	5
Suitable soil	4" gravel	3-5	7
Suitable soil	None	6+	10
Stabilized aggregate	None	3-5	13
Suitable soil	4" gravel	6+	14
Soil asphalt	None or single bit.	3-5	14
Stabilized aggregate	Single bit.	3-5	15
Stabilized aggregate	None	6+	18
Soil asphalt	None or single bit.	6+	19
Stablized aggregate	Single bit.	6+	20

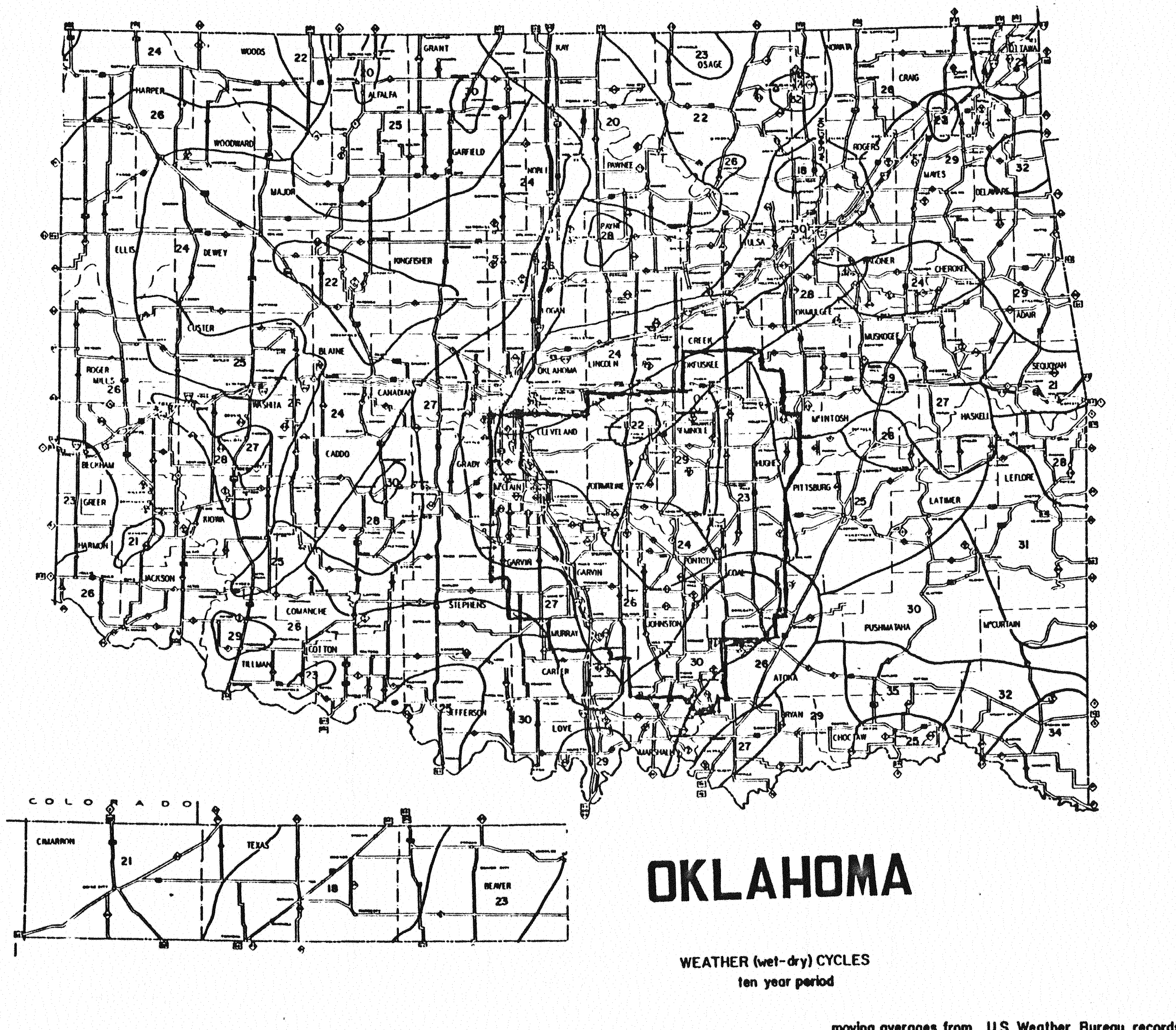

Figure 14. Climate factors.

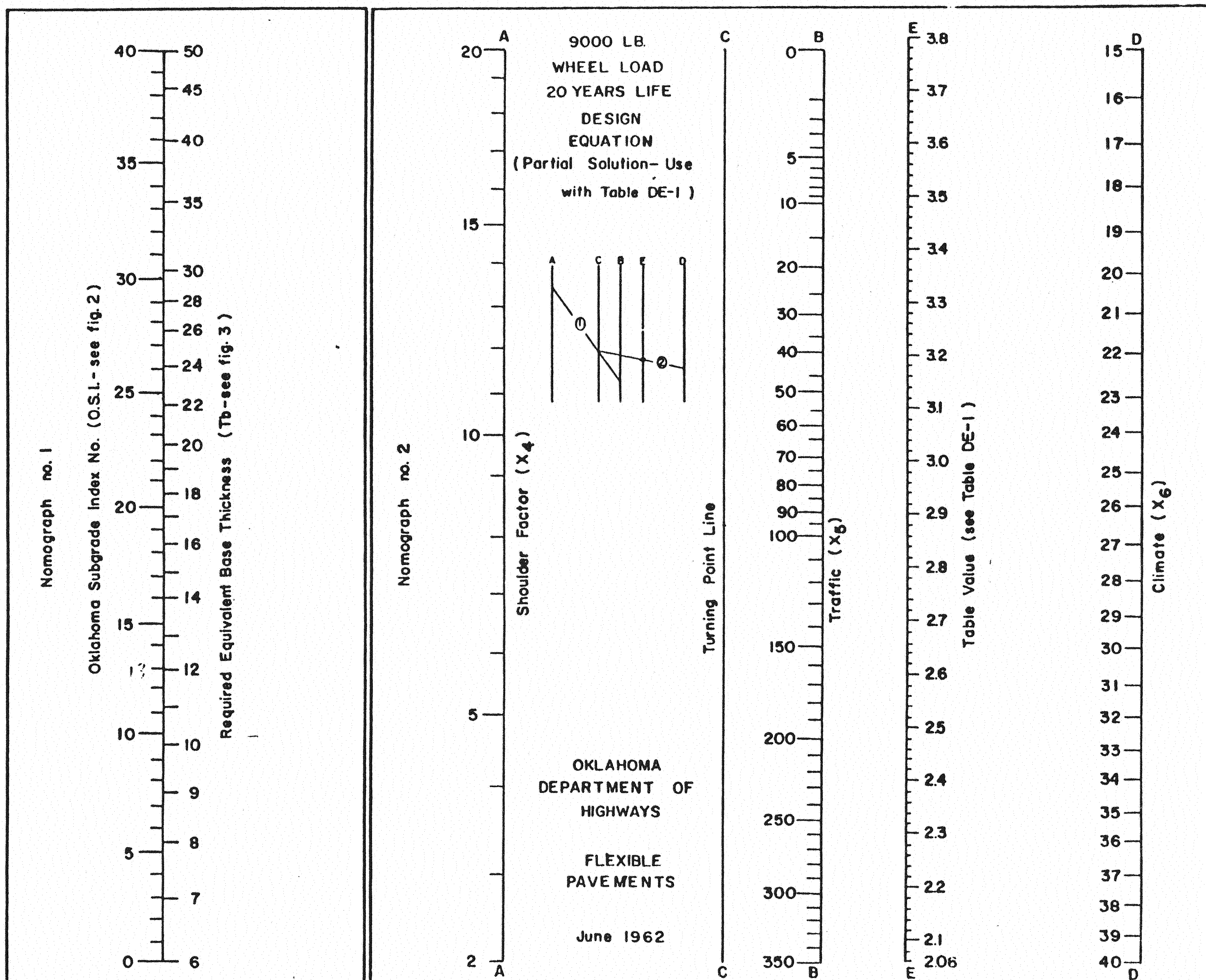

Figure 15. Nomograph for EBT adjustment by different factors.

Table 6. Design equation solution.

Correction (apply to T_b)	SHOULDER FACTOR (X_4) 2	3	4	5	6	7	8	9	10
-10	3.362	3.342	3.322	3.303	3.283	3.264	3.244	3.224	3.205
- 9	3.332	3.314	3.296	3.279	3.261	3.243	3.226	3.208	3.190
- 8	3.330	3.285	3.269	3.253	3.238	3.222	3.206	3.191	3.175
-7	3.268	3.254	3.240	3.227	3.213	3.199	3.186	3.172	3.158
-6	3.234	3.222	3.211	3.199	3.187	3.175	3.164	3.152	3.140
-5	3.199	3.189	3.179	3.169	3.160	3.150	3.140	3.130	3.120
-4	3.162	3.154	3.146	3.138	3.131	3.123	3.115	3.107	3.099
-3	3.123	3.117	3.111	3.106	3.100	3.094	3.088	3.082	3.076
-2	3.083	3.079	3.075	3.071	3.067	3.063	3.059	3.055	3.051
-1	3.040	3.038	3.036	3.034	3.032	3.030	3.028	3.026	3.024
-0	2.995	2.995	2.995	2.995	2.995	2.995	2.995	2.995	2.995
+1	2.948	2.950	2.952	2.954	2.956	2.958	2.961	2.962	2.964
+2	2.898	2.902	2.906	2.909	2.913	2.917	2.921	2.925	2.929
+3	2.844	2.850	2.856	2.862	2.868	2.874	2.880	2.886	2.892
+4	2.788	2.796	2.803	2.811	2.819	2.827	2.835	2.843	2.850
+5	2.727	2.737	2.747	2.757	2.766	2.776	2.786	2.796	2.806
+6	2.662	2.674	2.686	2.697	2.709	2.721	2.733	2.744	2.756
+7	2.592	2.606	2.619	2.633	2.647	2.660	2.674	2.688	2.702
+8	2.516	2.531	2.547	2.563	2.578	2.594	2.610	2.626	2.641
+9	2.433	2.450	2.468	2.486	2.503	2.521	2.539	2.556	2.574
+10	2.341	2.361	2.380	2.400	2.420	2.439	2.459	2.478	2.498
+11	2.240	2.261	2.283	2.305	2.366	2.348	2.369	2.391	2.412
+12	2.126	2.150	2.173	2.197	2.220	2.244	2.267	2.291	2.314
+13	1.996	2.022	2.047	2.073	2.098	2.124	2.149	2.175	2.200
+14				1.928	1.956	1.983	2.011	2.038	2.066

Table 6. Design equation solution (contd.).

Correction (apply to T_b)	SHOULDER FACTOR (X_4) 11	12	13	14	15	16	17	18	19	20
-10	3.185	3.166	3.146	3.126	3.107	3.087	3.068			
-9	3.173	3.155	3.137	3.120	3.102	3.085	3.067			
-8	3.159	3.144	3.128	3.112	3.097	3.081	3.065	3.049		
-7	3.144	3.131	3.117	3.103	3.090	3.076	3.062	3.048	3.035	
-6	3.128	3.116	3.105	3.093	3.081	3.069	3.058	3.046	3.034	3.023
-5	3.111	3.101	3.091	3.081	3.071	3.062	3.052	3.042	3.032	3.022
-4	3.091	3.083	3.076	3.068	3.060	3.052	3.044	3.036	3.029	3.021
-3	3.070	3.064	3.059	3.053	3.047	3.041	3.035	3.029	3.023	3.017
-2	3.047	3.044	3.040	3.036	3.032	3.028	3.024	3.020	3.016	3.012
-1	3.022	3.021	3.019	3.017	3.015	3.013	3.011	3.009	3.007	3.005
0	2.995	2.995	2.995	2.995	2.995	2.995	2.995	2.995	2.995	2.995
+1	2.966	2.967	2.969	2.971	2.973	2.975	2.977	2.979	2.981	2.983
+2	2.933	2.937	2.941	2.945	2.949	2.953	2.957	2.960	2.964	2.968
+3	2.897	2.903	2.909	2.915	2.921	2.927	2.933	2.939	2.944	2.950
+4	2.858	2.866	2.874	2.882	2.890	2.898	2.905	2.913	2.921	2.929
+5	2.815	2.825	2.835	2.845	2.855	2.864	2.874	2.884	2.894	2.904
+6	2.768	2.780	2.791	2.803	2.815	2.827	2.838	2.850	2.862	2.874
+7	2.715	2.729	2.743	2.757	2.770	2.784	2.798	2.811	2.825	2.839
+8	2.657	2.673	2.688	2.704	2.720	2.735	2.751	2.767	2.782	2.798
+9	2.591	2.609	2.627	2.644	2.662	2.680	2.697	2.715	2.733	2.750
+10	2.518	2.537	2.557	2.576	2.596	2.616	2.635	2.655	2.674	2.694
+11	2.434	2.455	2.477	2.499	2.520	2.542	2.563	2.585	2.606	2.628
+12	2.338	2.361	2.385	2.408	2.432	2.455	2.479	2.502	2.526	2.549
+13	2.226	2.251	2.277	2.302	2.328	2.353	2.379	2.404	2.430	2.455
+14	2.093	2.121	2.148	2.175	2.203	2.230	2.258	2.285	2.313	2.340
+15	1.932	1.962	1.991	2.021	2.050	2.079	2.109	2.138	2.168	2.197
+16						1.888	1.919	1.950	1.982	2.013

Table 7. Sensitivity of thickness adjustment on various factors

Factors			Thickness Adjustment (inch)
Shoulder	Traffic	Climate	
2	5	18 26 34	0 0 4
	100	18 26 34	0 4 8
	300	18 26 34	4 8 11
10	5	18 26 34	0 0 0
	100	18 26 34	0 2 7
	300	18 26 34	2 7 11
20	5	18 26 34	0 0 0
	100	18 26 34	0 0 7
	300	18 26 34	0 8 12

A close examination of this table indicates that the sensitivity of the shoulder factor in determining the thickness adjustment is negligible for combinations of large climate and traffic factors. This table shows that the thickness is inversely proportional to the shoulder factor for the combinations of smaller values of climate and traffic factors. This observation is realistic because the pavement thickness should be greater for a smaller shoulder width. These relationships between the thickness adjustment and shoulder factor for different values of climate factors with a traffic factor of 300 are shown in Figure 16.

Similar relationships between thickness adjustment with climate factor and traffic factor are shown in Figures 17 and 18, respectively. Figure 17 shows that thickness adjustment is very sensitive to the climate factor for different values of the shoulder factor. This figure also shows that this sensitivity is more significant for larger values of shoulder factors. Similar conclusions can be drawn from Figure 18 about the sensitivity of thickness adjustment on the traffic factor.

The Oklahoma Design Procedure is based on the value of OSI. The value of OSI is a function of liquid limit, plasticity index, and percent passing the No. 200 mesh for the subgrade soil. Previous studies have shown that these soil properties are influenced by the Resilient Modulus (2). Thus, by comparing the solid properties with the Resilient Modulus, a direct tie can be made between the Oklahoma Design Procedure and the new AASHTO Design Procedure.

DESIGN CHECK FOR PAVEMENTS

The eight projects sites including two rigid pavement sites, five flexible pavement sites, and one composite pavement site were redesigned using the Oklahoma design procedure. The purpose of this investigation was to determine whether all the requirements were met in determining the EBT during the original design. All the information for design was obtained from the Oklahoma DOT for the eight project sites. A series of

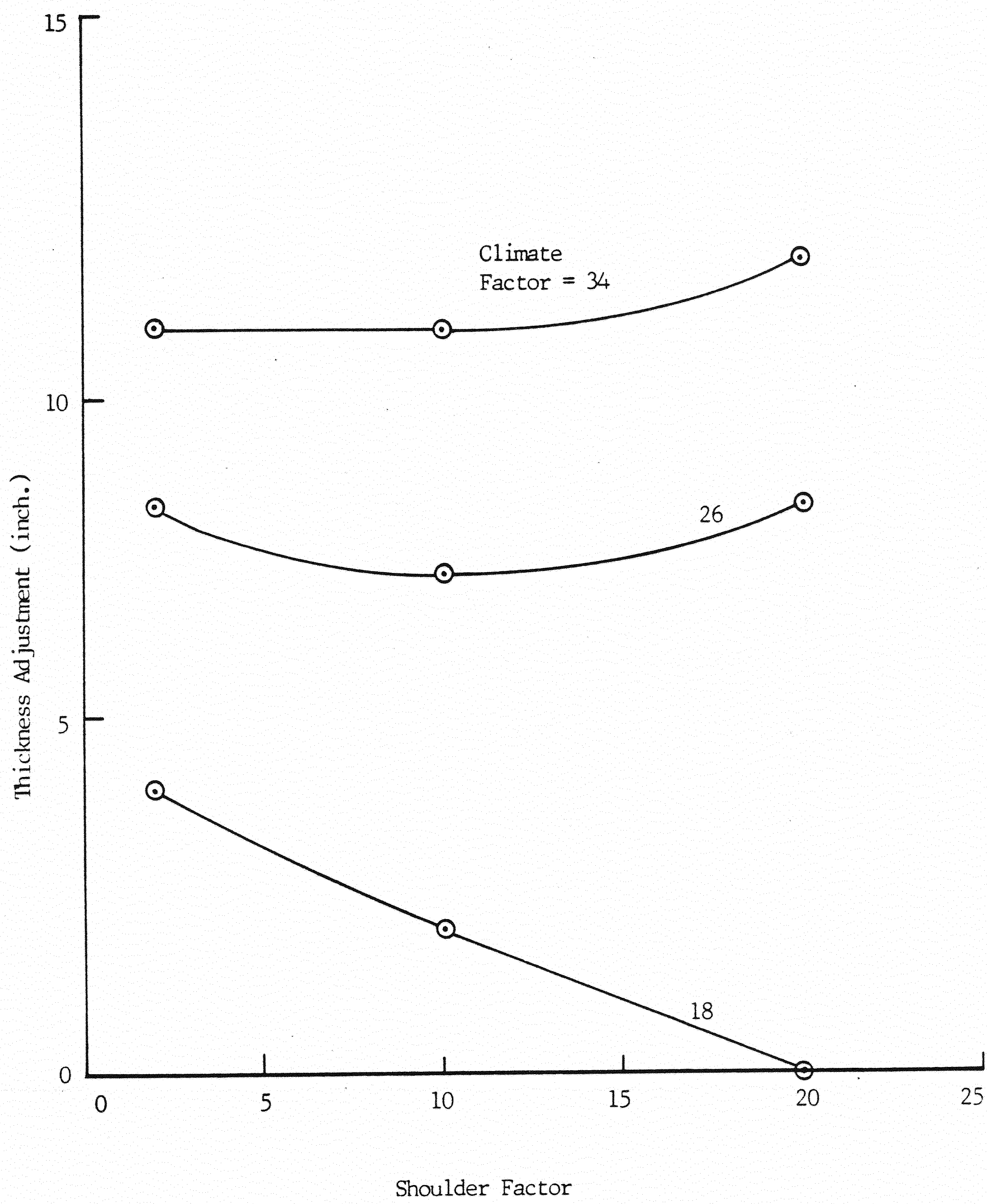

Figure 16. Sensitivity of thickness adjustment on shoulder for a traffic factor value of 300.

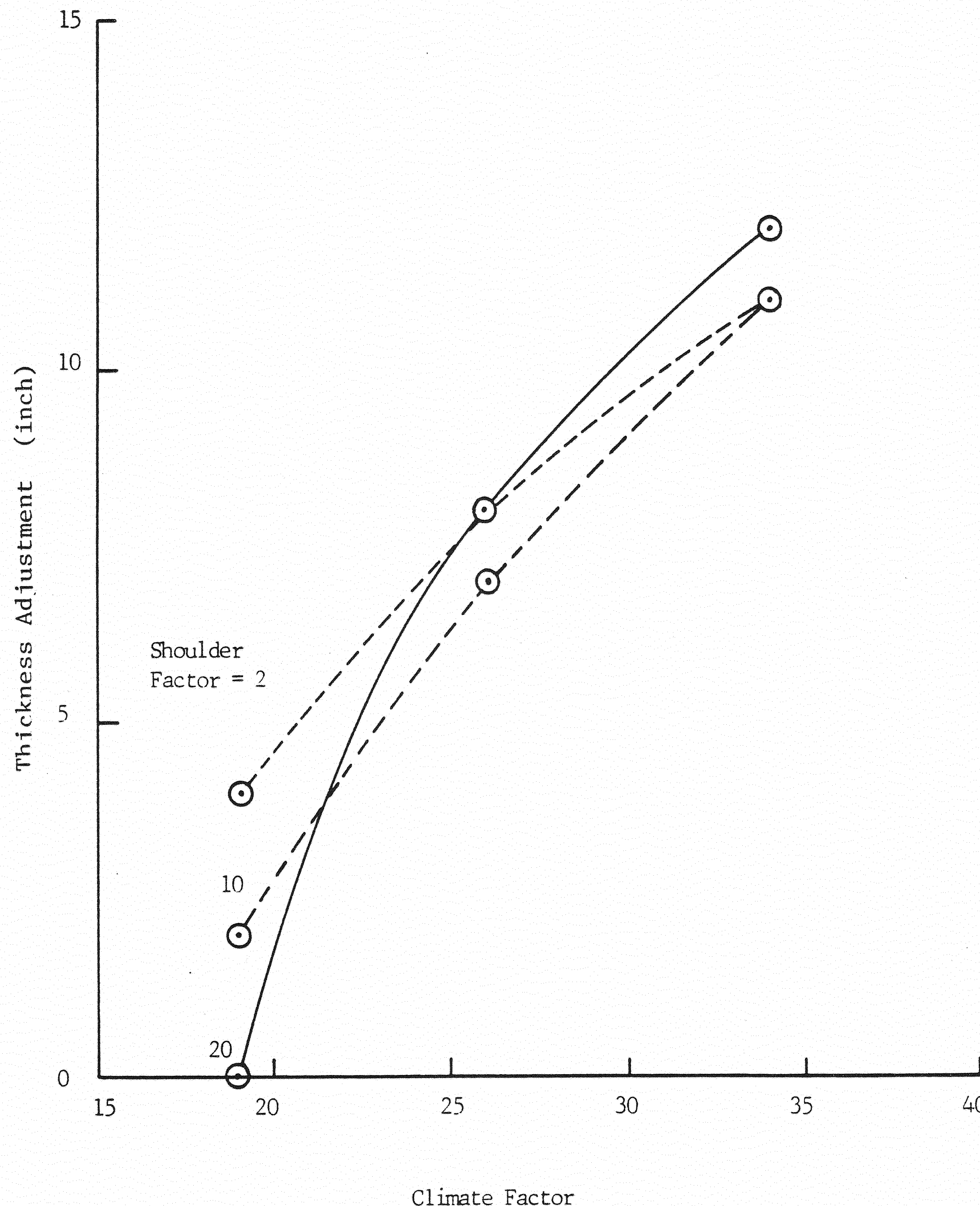

Figure 17. Sensitivity of thickness adjustment on climate factor for a traffic factor = 300.

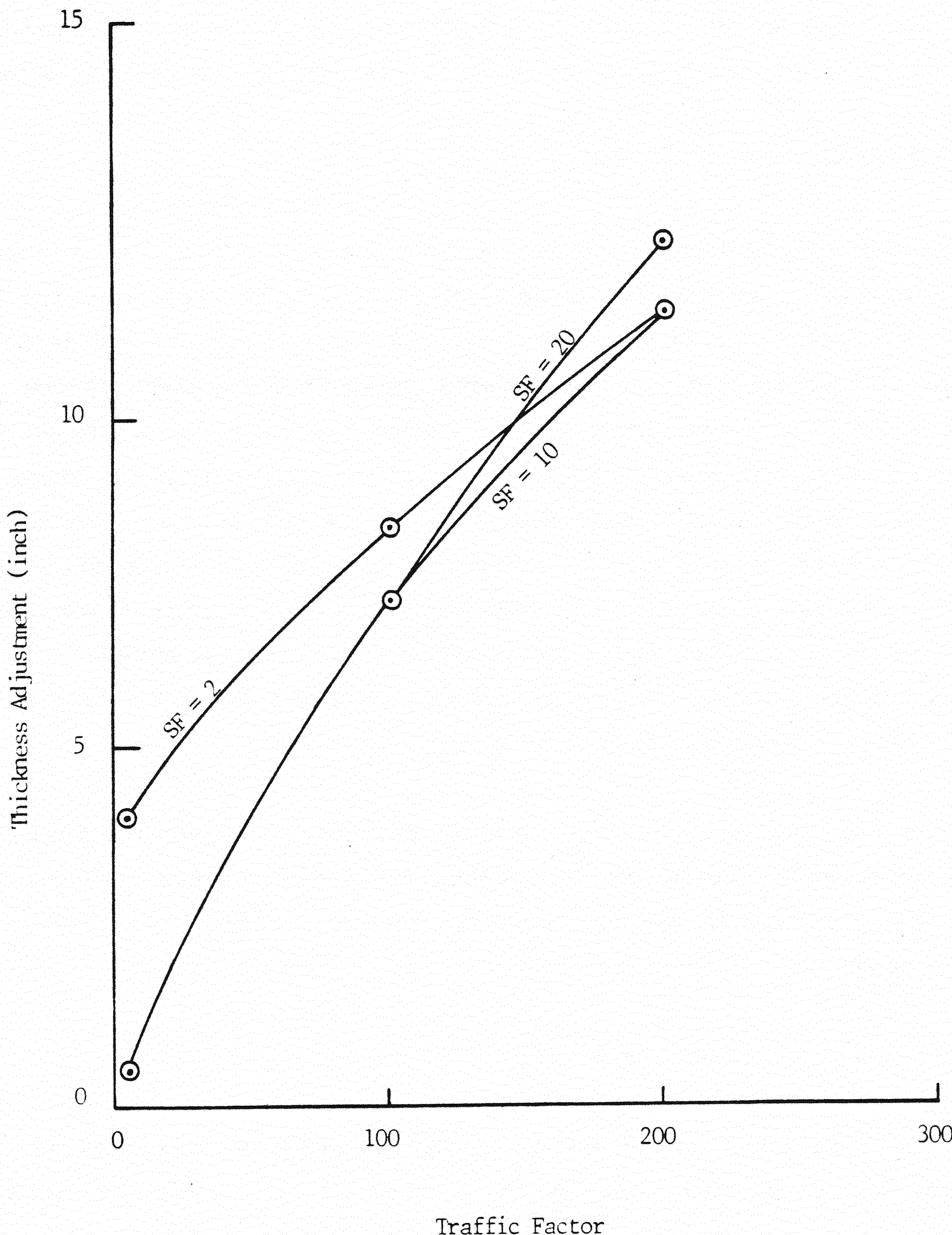

Figure 18. Sensitivity of thickness adjustment on traffic factor for a climate factor = 34.

computations lead to the conclusion that, the currently used design procedure was followed during the design of rigid and composite pavement sites. Similar conclusions can be drawn for the flexible pavement sites.

ODOT PAVEMENT MANAGEMENT PROCEDURE

Pavement management is an organized procedure used to identify and implement strategies at various management levels. These optimum and/or priority strategies are derived through clearly established and defined procedures. The concept of a comprehensive pavement management system is illustrated in Figure 19. This figure shows the three major types of activities, namely Global or Network activities, Local or Project activities, and Feedback or Update activities. The network level provides for a rational matchup of funds (resources) and needs (pavement distress) i.e., good planning. The project level provides a good design analysis covering all the costs incurred by maintenance, initial construction, and user costs during the design life. The monitoring provides a data base for correcting the future designs.

At present, ODOT does not have a well defined pavement management system in practice. Maintenance and rehabilitation procedures currently practiced by ODOT include several standard techniques for both rigid and flexible pavements.

COMPARISON OF ODOT & AASHTO DESIGN PROCEDURES

The AASHTO pavement thickness design procedure has been revised (2) in 1985 and has been submitted to the states for review and possible

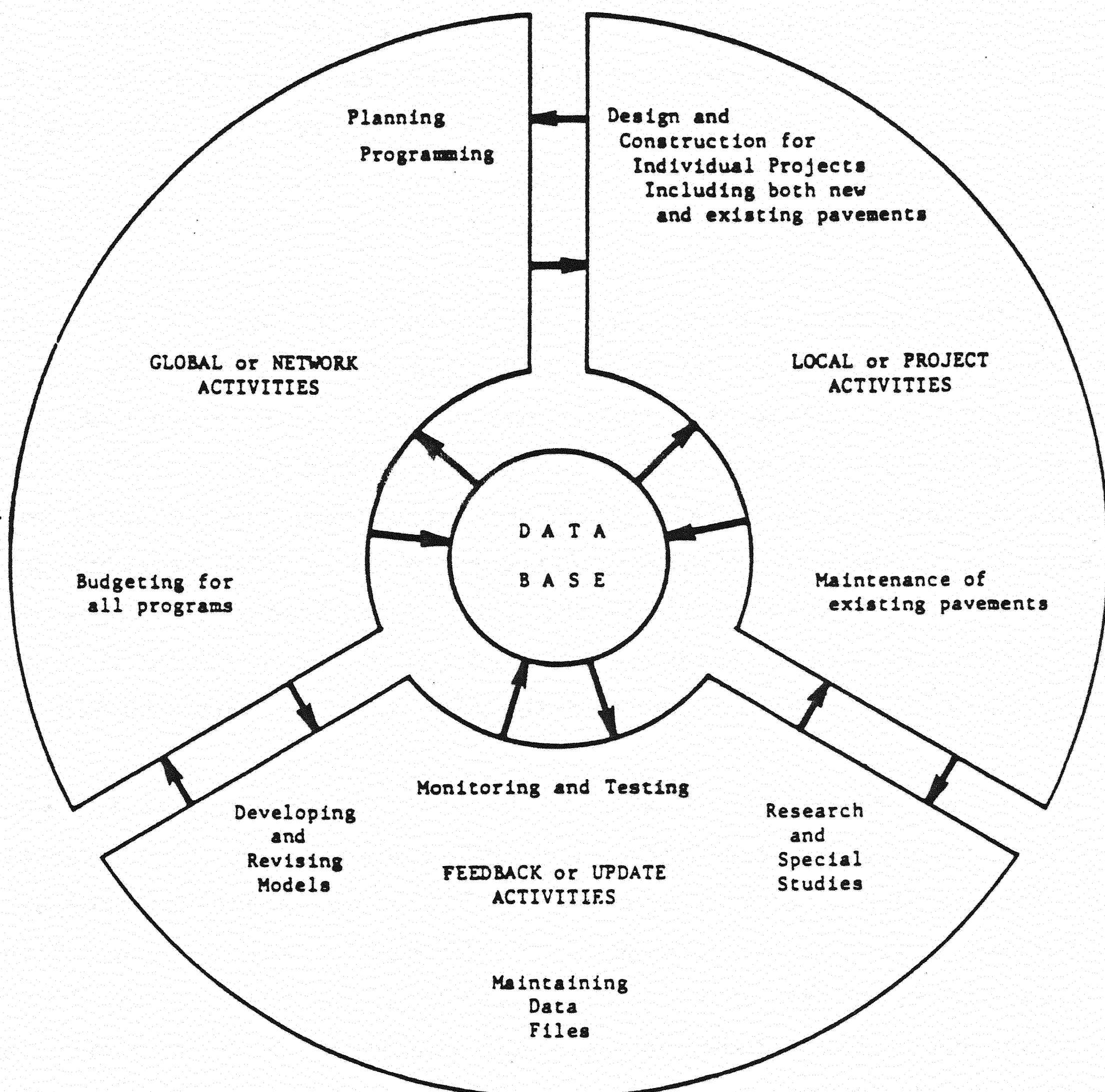

Figure 19. Activities of a Pavement Management System

approval. In this section, the procedure is compared to the ODOT pavement design procedure. A direct comparison between these two procedures was not possible due to the two different approaches followed. The structural numbers (SN) for all the five flexible pavement sites were determined by following the AASHTO procedure using the subgrade moduli obtained from the non-destructive evaluations and testing. In these computations, a reliability (R) of 99 percent and design serviceability loss (PSI) of 2.5 was used. It was assumed that the effective roadbed soil resilient modulus (M_R) was equal to the measured moduli and its value did not change over the service period.

The SN value for each existing flexible pavement site was computed by using the equivalent base thickness (EBT) and was designated as the ODOT structural number. These structural numbers obtained from both AASHTO and ODOT are shown in Table 8. This table shows that for all five of the flexible pavement sites investigated, the structural number obtained for the existing pavements, originally designed by using ODOT pavement design procedure, are higher than those obtained by using the AASHTO pavement design procedure. The differences between these two structural numbers are from 0.83 to 2.83 with a maximum for Site No. 6 as shown in Table 8.

The above observation leads to the conclusion that, as far as the structural strength is concerned, the ODOT design is on the conservative side. This is indicated by comparing the higher SN values required by the ODOT procedure with those obtained from the AASHTO design for all five of the flexible pavement sites. This supports the conclusion that the premature distresses observed were not caused by inadequate structural strength of the pavement.

The above procedure of determining the SN value using the AASHTO design method is based on the moduli of the subgrade. An error in determining this moduli from deflection measurements will result in an error in the SN values. In order to investigate the range of values of SN that could result from subgrade moduli, the data for Site No. 2 were used

Table 9. Comparison of SN required from ODOT and AASHTO method using 99 percent reliability and $\Delta PSI = 2.5$.

Site No.	E (psi)	W_{18} (million)	SN		ΔSN^{*}
			AASHTO	ODOT	
2	23,600	8.80	3.50	4.40	0.90
3	23,800	5.84	3.20	4.38	1.18
4	27,200	6.83	3.10	4.40	1.30
6	24,900	8.18	3.35	6.18	2.83
7	14,000	1.95	3.25	4.08	0.83

* $\Delta SN = SN_{ODOT} - SN_{AASHTO}$

as a representative site for flexible pavement. Assuming t-distribution for the subgrade moduli, the range of values for the $\overline{SN}$ is defined by Eq. 8.

$$SN = \overline{SN} \pm \left[\frac{\sigma_{SN}}{\sqrt{N}} \right] t_{\alpha} \qquad (8)$$

in which σ_{SN} = standard deviation of SN, N = number of data set, $\overline{SN}$ = mean value of SN, and t_{α} = value of t - distribution. The quantity in the bracket of Eq. 8 is the estimated standard error of the mean. This standard error of the mean was computed by defining

$$\sigma_{SN} = (CV)\ \overline{SN} \qquad (9)$$

in which CV = coefficient of variation of the structural number. The assumption was made that the CV of the moduli and SN are the same. Using $\overline{SN}$ = 3.5 for Site No. 2 the two limiting values of SN for a 50 percent confidence level are 3.12 and 3.87. Similar values for 90 percent confidence level are 2.57 and 4.43. Compared to these values, the $\overline{SN}$ for ODOT design is 4.4. Define the asphalt concrete pavement thickness variation (ΔD_{AC}) as a function of SN variation (ΔSN) by

$$\Delta D_{AC} = \frac{\Delta SN}{0.40} \qquad (10)$$

where 0.40 is the assumed value of asphalt concrete layer coefficient (5). Using Eq. 10, the range of thickness variation for 50 percent confidence level is 1.88 and for 90 percent confidence level is 4.65.

CHAPTER 5

RECOMMENDATIONS

The current ODOT guide (4) for pavement thickness design was carefully reviewed to determine its limitations, if any. This was accomplished by comparing the guide with the recently revised AASHTO guide (2) for designing pavement structures, non-destructive evaluation of pavement structures, laboratory testing of pavement materials, diagnostic evaluation by a team of experts and interviews and evaluation by the ODOT personnel. From these investigations, the reasons for premature failure of the six pavement sites were determined and a basis was developed for recommending changes of the ODOT pavement thickness design guide. These recommendations are divided into three general groups and discussed in this chapter. Although the recommended changes are not exact and ready for direct incorporation into the guide, they are of significant importance in the design of future roadways for successful performance of the pavements in the Oklahoma highway system.

DESIGN PROCEDURE

The following recommendations are proposed for consideration by ODOT for incorporation into the guide currently used for designing the pavement thickness.

1. The design procedures should be based on cumulative expected 18-kip equivalent single axle loads (ESAL) during the analysis period. Mixed traffic may be converted to 18-kip ESAL units by using a procedure recommended by AASHTO (2). The predicted 18-kip ESAL should be distributed by direction and then by lanes.

2. An analysis period of more than 20 years may be considered in the design. This longer performance period may be more suitable

for the evaluation of alternative long term strategies based on life cycle costs.

3. The reliability concept should be introduced in the design guide. This insures that the various design alternatives will last the analysis period. Reliability accounts for the chance of variations in both traffic and performance prediction.

4. Load transfer devices should be provided at the joints of rigid pavements, concentrated in the wheel paths. This will minimize the faulting at rigid pavement joints and provide a longer pavement life.

5. The procedure should include a detailed drainage design to expedite the removal of water runoff. This measure will help maintain a low moisture content in the subbase and subgrade and thus reduce several material related problems.

6. A new material characterization test should be used in terms of Resilient Modulus so that a rational testing can be performed, thus permitting the implementation of mechanistic design procedures in the future and a correlation with the proposed revised AASHTO guide.

7. Background experience can be obtained in resilient modulus values by using deflection measurements in predicting material properties. Either the Dynaflect or FWD can be used for this purpose.

8. For new design, a correlation between the OSI and the Resilient Modulus test will permit a direct application of the proposed revised AASHTO Guides to compare with the ODOT procedures.

9. The deflection based overlay design procedures in the proposed AASHTO Guides should be considered for rehabilitation design.

MATERIAL SPECIFICATIONS

The following recommendations are proposed for incorporation into the currently used material specifications to avoid the stripping of aggregates in the base and subbase materials of the flexible pavement structures.

1. Mix design requirements should be re-evaluated. This can be accomplished by the split tensile test, creep test, and Lottman test.

2. Hydrated lime should be used to correct stripping and water sensitivity problems. Chemical additives may also be used, provided their effectiveness is evaluated in the laboratory prior to use.

3. Harder asphalt should be used on heavily trafficked highways having thick asphalt concrete pavements. For these types of highways, consideration should be given to using a higher percentage of crushed aggregate.

PAVEMENT MANAGEMENT ACTIVITIES

ODOT does not have a well defined pavement management system in practice at the present time. It is recommended that a pavement management and evaluation system be developed for the Oklahoma highway system. PMS should be developed both at a network and local levels. The network PMS may be used to distribute the resources to the needs in a rational manner. The project level PMS will help in the design of better pavements in the future and should reduce maintenance costs. In the meantime, the following maintenance procedures are recommended for protecting the pavements from possible distress.

1. Joints should be sealed and maintained to protect the underlying layers from moisture.

2. Slabs having voids underneath should be subsealed.

3. Random cracks which require sealing should be identified and sealed.

4. Slabs requiring full depth patching or replacement should be identified and repaired.

5. On concrete pavements experiencing joint faulting with heavy traffic, consideration should be given to installing load transfer devices.

6. A seal coat should be applied on flexible pavements experiencing stripping in the underlying layers due to the presence of moisture susceptible aggregates. This will prevent moisture entry to the problem layers. Also cracks on the flexible pavements appearing during winter should be sealed with rubber asphalt.

7. The upper layer of asphalt concrete of a specified thickness should be removed for the rutting flexible pavements. This layer should be replaced by a high density mix using AC 40 as binder. Lime slurry should be used in the mix to improve moisture resistive properties.

CHAPTER 6
PROPOSED RESEARCH

Recommendations proposed in Chapter 5 are qualitative in nature. These require further investigation before incorporating them into the currently used pavement thickness design, material specifications and pavement management. A detailed description of the proposed research program and a plan for achieving this goal is presented in this chapter.

RESEARCH PROGRAM

On the basis of the recommendations made for upgrading the currently used pavement thickness design, material specifications and pavement management, the following research activities are proposed.

1. A state-wide investigation should be conducted to identify the areas in which stripping has occurred. As a part of this investigation criteria should be developed for identifying potentially stripping aggregates and conditions where they may and should not be used. As an example in Chapter 2, values were proposed for percent asphalt retained, accumulated traffic and rainfall based on the limited sections studied i.e. seven. This investigation would be limited to condition surveys and material testing and would not require deflection testing, PSI readings, etc.

2. On the basis of this investigation, specifications should be developed for testing requirements. Also, consideration should be given to climatic, and traffic factors.

3. A data base should be developed for both global and local levels. This data base should contain information encompassing planning,

budgeting, design, construction and maintenance. Using this data base, a pavement management system should be implemented for the Oklahoma DOT.

4. A task should be undertaken for the revision of the currently used pavement design procedures and material specifications to incorporate some of the desirable features of the proposed revised AASHTO pavement guides as outlined in Chapter 5.

5. A deflection measuring program may be used to obtain data for backcalculating the resilient modulus and then may be correlated with the OSI for future comparisons of the ODOT procedure with the AASHTO Guides and future mechanistic design methods.

RESEARCH PLAN

In the previous section, a series of tasks were outlined for a research program. The following is recommended as possible projects and their priority.

1. Tasks 1 and 2 should be combined into a project to characterize the stripping problem and, thus minimize rutting on future pavements designed by ODOT.

2. Tasks 4 and 5 should be combined into a project to improve the ODOT procedures using the revised Guides and mechanistic design concepts.

3. The information from the first two projects will develop a data base that can be used as a starting point for developing an ODOT pavement management system.

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