EVALUATION OF OKLAHOMA PAVEMENT DESIGN PROCEDURES

Phase I Report

REPORT NO. 84-60

Submitted to:

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16. Abstract
This report covers collection of pavement conditions and other pertinent performance data for 6 representative pavement sites that have failed prematurely and 2 sites which have performed as intended. Interviews of Oklahoma DOT personnel, review of records, inspection by an expert diagnostic team together with a field data collection program are also included.

Based upon this data as well as laboratory testing and analysis of the deflection testing performed, an assessment of the causes for early failures at the representative sites are made. Recommendations for changes in design and construction practices to prevent future premature pavement failures are included.

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Compendium of Informations For Phase 1
Report No. 84-60

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EXECUTIVE SUMMARY

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 term 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 detail 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.

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CHAPTER 1
INTRODUCTION

The Oklahoma DOT's basic method of pavement design (Ref 1) has been in use for some 20 years. Since implementation in 1962, many changes have occurred in many areas which warranted a review and evaluation of their current methods and procedures. In the analysis area, numerous tools such as computer programs, stress models, etc. have become available. In addition, traffic volumes, truck percentages, legal load limits, truck tire pressure and many other design factors which influence performance have increased. Also, construction materials, methods, machinery, and practices have changed considerably.

At the present time, Oklahoma's Interstate System is nearing completion, and those sections constructed first are being rehabilitated or reconstructed. Several sections of pavement, both flexible and rigid, have undergone premature deterioration or failure. Since available funds for highway construction are very limited, the Department felt it imperative that pavement design and management practices assure a full design life with minimal maintenance expenditures.

This research project was initiated with these factors in mind. A limited set of projects were selected to determine if any factor could be singled out to help determine reasons of poor and good performances of flexible, rigid, and composite pavements. Out of the total eight pavement sites selected, there were two rigid, five flexible, and one composite pavement. The age of failed pavements ranged from two to fifteen years for flexible pavements and three years for one rigid pavement. The age of good pavements are six and seventeen years for flexible and rigid pavements, respectively.

OBJECTIVES OF RESEARCH PROJECT

The objectives of this research project are as follows:

1. To determine the reasons for premature failure of six pavement sections which are considered to be representative of projects in the Oklahoma system, and to relate the reasons to possible deficiencies in the pavement design procedure or management practices.

2. To review the present pavement design selection and design procedures and make appropriate recommendations for changes. The review shall include new construction, rehabilitation, reconstruction, and overlay of existing pavements. It shall also determine the possible need for pavement design life spans in excess of 20 years based upon a proper economic justification.

RESEARCH APPROACH

To assist the reader in establishing an overall project approach, a description of the projects selected for study is provided and this if followed by an explanation of the research approach.

Description of Pavement Projects

The pavement sections selected for investigation to determine the reasons for severe distress and/or failure are designated as Sites 1, 2, 3, 4, 5, 6, 7, and 8. Table 1.1 presents background information as to location, age, and pavement type. Figure 1.1 shows the 8 sites on an Oklahoma State Highway map. It is obvious that these study sites cover a very wide regional area of the state, and a range of pavement ages (2-17 years).

All sites were constructed on the basis of the same pavement design procedure. Sites 1 and 7 designated as good sites are performing as intended, but the other sites have varying degree of distress. Site 6 was originally considered a good section but it was learned during the study

Table 1.1. 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.					

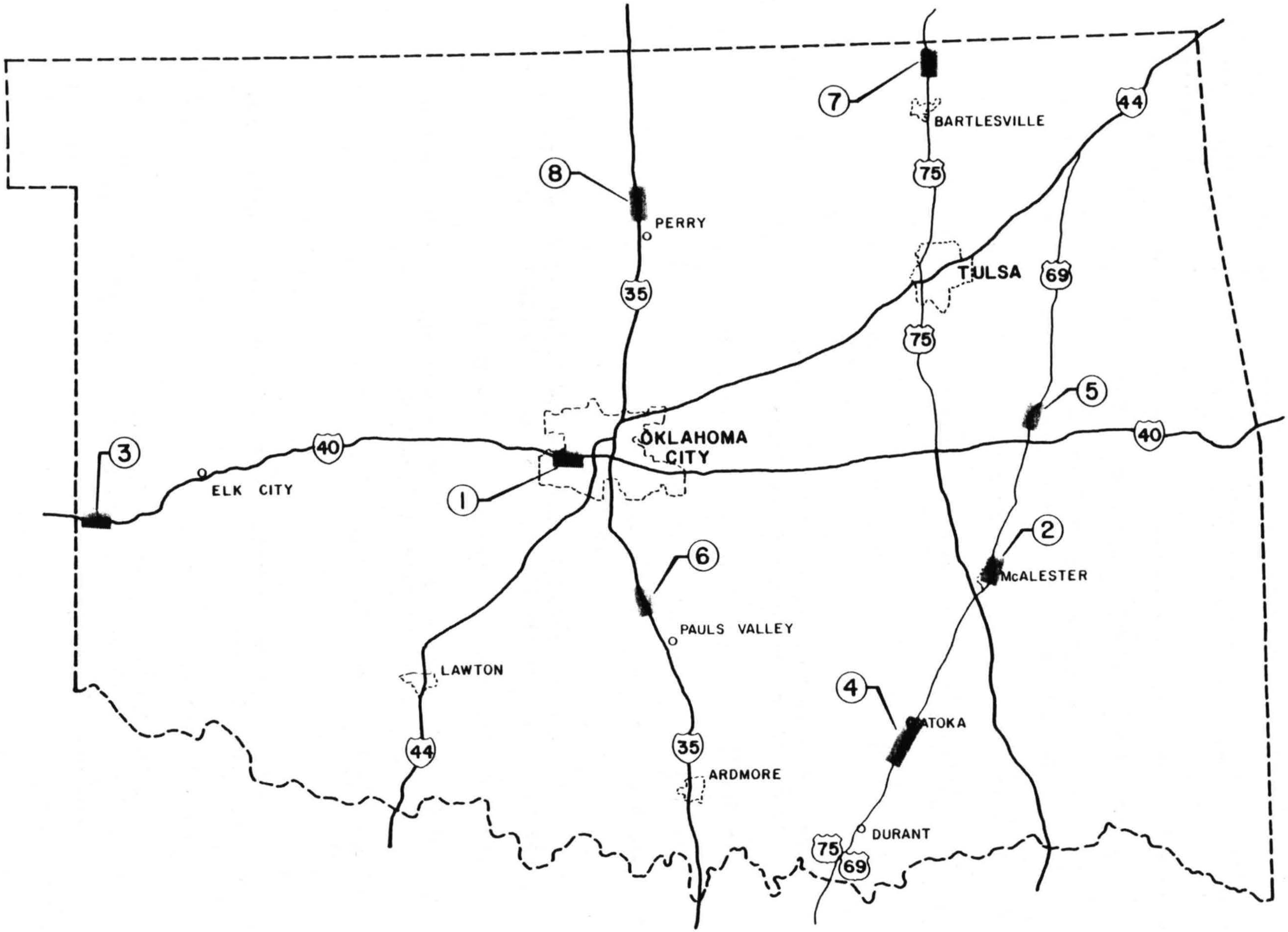


Figure 1.1. Locations of eight study sites in Oklahoma.

that this project had been rehabilitated by a 2" ACP overlay and an open graded friction-course. Additionally, rutting was occurring in the wheel paths.

Scope of Research Project

This investigation was conducted for all the three pavements types, namely rigid, flexible, and composite. Investigation procedures included interviews and evaluations by the Oklahoma DOT personnel, diagnostic evaluations by an expert team, non-destructive field testing and evaluations, laboratory testing and evaluations. On the basis of these investigations, the reasons for premature failure of the pavement projects was determined. A detailed review of the departmental pavement design procedures and management practices is being conducted by ARE Inc.

OBJECTIVES OF REPORT

The objective of this report is to comply with the requirements of Phase-I which specifies that a comprehensive interim report be submitted covering the data collection, its analysis and the subsequent testing activities. Also, the probable failure mechanisms present in each of the six "failed" sections are to be identified and attributes related to the good performing pavements is to be evaluated.

SCOPE OF REPORT

This report is a documentation of all the findings relative to the reasons for distress in each of the failed sections. The following four chapters are devoted to the detailed description of the various investigations conducted by ARE Inc. The concluding two chapters summarize the reasons for the distress of the six pavement sites and satisfactory performance of the two pavement sites.

CHAPTER 2

INTERVIEWS AND EVALUATIONS

In order to obtain background information in relation to the eight pavement sites and to prepare for the planned field investigations, a series of interviews were conducted with the Oklahoma DOT officials. Two groups were interviewed: (1) a headquarters group; designated as the research project selection committee, and (2) engineers for the respective divisions in which eight pavement sites were located.

INTERVIEW OF RESEARCH PROJECT SELECTION COMMITTEE

The research project selection committee consisted of the following named personnel:

Tim Borg	Project Engineer
Key Boyd	Director of Secondary Roads
Ed Cuaderes	Pavement Design Engineer
R.B. Hankins	Rural Design Engineer
C. Dwight Hixon	Research and Development Engineer
Monty C. Murphy	Assistant Director - Planning and Research

A series of questions were presented to the research project selection committee during the group interview as shown in Appendix A. The results of this interview are presented in this section, and represent the consensus of the committee at the time of the interview.

Results of the Interview

1. The sections were selected on the basis of performance, i.e., performing as expected or with premature distress and all sites were four lanes.

2. The major observed distresses are rutting, thermal cracking, stripping aggregates, "D" cracking of concrete pavement is becoming a maintenance problem. There is a difference in the state by regions, i.e. east and west have different geological formations, different climates and rainfall.

3. Condition surveys and Mays Meter values on US-69 (Site 5) are available along with a special investigation report. Soil surveys and Benkleman beam results are also available for Sites 3 and 6. Traffic volumes, design manuals, plans, specifications, and needs study information was provided at the meeting.

4. Mr. Ed Cuaderes, Office of Pavement Design, provided a description of the pavement design procedures together with manuals etc. Generally, all projects in Oklahoma are built in stages with grading and drainage first. The next phase is pavement onstruction that requires a soils study together with an estimate of future traffic to arrive at a pavement design.

5. The objective of this research project is to determine if inadequacies exist in their pavement design procedures and management practices.

INTERVIEW OF DIVISION ENGINEERS

The second group was composed of the division engineers and other personnel responsible for construction of the projects selected for the various test sites. During each of the failed test site visits, six questions were posed to the division engineers as shown in Appendix A. The only question asked about the two good sites was - Why are the present site performing well? Collective answers and comments for each test site are also given in Appendix A. The following paragraph summarize the findings from the interview of Division Engineers.

Generally two types of distress were observed by the division engineers on the flexible pavement sites which have failed prematurely. All these sites have shown rutting and varying degrees of shoving. In addition to this, stripping was also observed at Site No. 4. Among the two rigid pavement sites investigated, one (Site No. 5) showed joint faulting and pumping. The observed failures in the only one composite pavement site investigated were rutting, shoving, and reflection cracking in the asphalt layer.

The division engineers felt that the materials of construction are mainly responsible for the early failures of the flexible pavement sites investigated. According to their opinion, the quality of asphalt used was inferior, the stability of the sand asphalt is too low, and the stripping of asphalt is attributed to the water entering from the top and moisture not being removed by dryer drum during construction. This group of engineers also believe that the use of open graded mix, overloading of highways beyond the designated capacity, and inadequate design procedures are partially responsible for the premature failures of the flexible pavement sites.

The reasons for unsatisfactory performance of the rigid pavement site are mainly associated with the failures of the base and subgrade. According to the division engineers, higher deflections are caused due to the softening of the clay and shale by surface and subsurface water. They also blamed the inadequate stability requirements during construction, which resulted in a weak sand asphalt base.

The division engineers familiar with the site which represented the premature failing of a composite pavement, (Site No. 8), reasoned that the mixture was the main cause of failures. According to their opinion, the open graded friction course used holds water and causes stripping. They also felt that too much asphalt was used in the mix during construction.

The reasons for the satisfactory performance of Site No. 7, which represented the good flexible pavement, are primarily due to the good materials, construction, and design procedure. The division engineers also believe that strong roadbed soil is also responsible for the satisfactory performance. Similar reasonings were given by the division engineers of Site No. 1 for the satisfactory performance of this rigid pavement site.

SUMMARY

Interviews of the project selection committee and the division engineers of the eight investigation sites, along with their evaluations are summarized in this chapter. The pavement sections selected for investigation represent a wide range of environmental conditions in Oklahoma, pavement age, and pavement types. Most of the observations in failures of asphalt pavements pertained to material problems, and the thickness of the layers was not cited as a reason for unsatisfactory performance. Difference in opinion exists on joint spacing of rigid pavements. Difference in opinion also exists about the adequacy of the design procedure.

CHAPTER 3

DIAGNOSTIC EVALUATIONS BASED ON FIELD OBSERVATIONS

This chapter summarizes observations from a field trip with the objective of evaluating the performance of the eight pavement projects in the Oklahoma highway system by a diagnostic survey team. In this chapter, background information on the diagnostic survey is provided, followed by the observations, and then a summary section.

BACKGROUND INFORMATION

The evaluations were made by Dr. B. Frank McCullough and Fred Finn of ARE Inc. Mr. Tim Borg of the Oklahoma Department of Transportation organized the trip, participated in the survey, and provided valuable information pertinent to each project. The evaluations were made on July 30, 31, and August 1, 1984. The weather during this period was warm to hot (75°F in the morning to 90°F in the afternoon); no significant amount of rain had been reported in the area for at least 30 days.

Background information was limited to project plans provided by ODOT, and field trip evaluations provided by Mr. John Nixon of ARE Inc. The Nixon report was based on information obtained during the field testing of each project and included input obtained from discussions with ODOT personnel familiar with each project.

The visual evaluations were made at random locations along the project alignment. At each location, a detailed examination (walking) was made for approximately 300-500 feet. A minimum of three random stops were made in each direction of the designated sections. Depending on the pavement condition, additional stops were made in order to obtain a representative sample of the project. Ride ratings (PSR) were estimated between stops at a speed of approximately 55 mph.

SUMMARY OF FIELD OBSERVATIONS

A summary of the results of the pavement condition survey made at the time of deflection testing is shown in Table 3.1. Detail results of the pavement condition survey by McCullough and Finn for all the eight sites are shown in Appendix B and the reader is referred to it for a more encompassing view of the performance. Table 3.2 is an evaluation of stripping by the project staff based on the examination of core samples from the projects.

Based on these observations, the major problem in the performance of the asphalt pavements is plastic deformation as manifested by rutting in the wearing surface. Only one section, US-69 (Site 4), in Atoka County, exhibited a significant amount of alligator (fatigue/traffic related) cracking. The section on U.S. 69, north from McAlester (Site 2), had been maintained (repaired) by local maintenance forces and was difficult to evaluate. However, notes made by John Nixon during the sampling and testing phase indicated the asphalt was rutting and shoving with some cracking.

The observed incidence of pumping in the asphalt surface was limited, but was observed on the Atoka project and the McAlester project (Sites 2 and 4). Little or no rain had occurred in the vicinity of the project in over 30 days.

The primary type of distress observed on the portland cement concrete pavements was faulting at the transverse joints. The faulting on Site No. 5 (Failed Section) is excessive and minor faulting is evident on Site No. 1 (Good Section). The ride quality (PSR) was somewhat low on the PCC pavements due to the faulting, although the overall rating is considered good; i.e., considering the ride quality and the physical conditions of the pavement.

Table 3.1. Summary of Pavement Condition for Each Site.

Pavement Type	Site	Assigned[a] Condition	PSR[b]	General Observations
Flexible	2	Failed		Large amount of patching. Severe rutting and longitudinal cracks. Some ravelling, surface wear and transverse cracking.
	3	Failed	3.5-4	Severe rutting and small amount of surface wear.
	4	Failed	3-3.5	Large amount of ravelling and rutting, longitudinal, transverse, fatigue, and block cracking present.
	6	Failed	3.5-4	Large amount of rutting throughout.
	7	Good	4-4.5	Small amount of rutting.
Rigid	1	Good	4-4.5	Good condition overall. Some slight spalling and faulting. Fair edge joint and joint seal.
	5	Failed	2.5-3	Faulting severe in southbound and slight in northbound lanes. Poor ride quality. Poor edge joint but fair seal condition.
Composite	8	Failed	2.5-3.5	Large amount of rutting, shoving, and relfective cracks througout. Longitudinal cracks present.

(a) Condition status assigned by Project Selection Committee.
(b) Estimates and observations made by Finn and McCullough.

Table 3.2 Evaluation of Stripping Based on Examination of Project Cores as Received.

Site No.	Project	Description of Cores for Stripping	Performance
2	U.S.-69 Pittsburg County	A significant amount of stripping noted	In-service since 1982, section currently under repair
3	I-40-1 Beckham County	No significant amount of stripping noted	In-service since 1973, rutting up to 1 inch based on visual observation
4	U.S.-69 Atoka County	No significant amount of stripping noted	In-service since 1981, rutting up to 1.5 inches based on visual observation
5	U.S.-69 McIntosh - Muskogee Counties	Stripping noted in bituminous base for PCC	In-service since 1981, faulting up to 0.75 inches
6	I-35 McClain County	A significant amount of stripping noted	In service since 1969, overlay in 1979, rutting up to 0.75 inches
7	U.S.-75 Washington County	No stripping noted	In-service since 1978, rutting estimated at 0.25 inches
8	I-35 Noble County	Estimated stripping at 30% in Type C mix	In-service since 1980, rutting up to 1 inch based on visual observation

The Washington County project on U.S. 75 (Site 7), south from the Kansas state line, was cited as a good performing asphalt type pavement, which it was. However, the traffic on this section is relatively low (ADT 4850), and the pavement structure thickness was relatively large (18.5 inches) compared with the other asphalt surfaced pavements.

CONCLUSIONS

Based on field observations, it is concluded that the problems in flexible pavements are related to a general lack of stability or the ability to resist plastic deformation under service conditions. This problem is probably compounded in some cases by low tensile strength properties when the mixes are wet and subject to the dynamic effects of traffic.

It is pertinent to note that rutting occurred in projects with and without hot mix sand asphalt. Also, a significant amount of rutting is believed to be occurring on I-35 in Noble County (Site 8) on a project that has only 2.75 inches of asphalt concrete over an old PCC pavement.

After returning to ARE offices in Austin, visual examination of project cores was undertaken to subjectively evaluate the stripping characteristics of the asphalt concrete before subjecting them to laboratory conditioning, i.e., vacuum saturation for the modified Lottman Test. A summary of observations are also given in Table 3.2.

While the observations summarized in Table 3.2 are somewhat limited, the occurrence of stripping was noted in five out of seven of the projects for which cores were available. Stripping was noted even in the asphalt stabilized subbase beneath the PCC pavement at Site no. 5. It is also of interest to note that one of the non-stripping sites is Site no. 7 (Good Section), and the other one is Site no. 6, which was originally classified as good site. Therefore the common thread running through the projects is the stripping of one of the layers. With the stripping action, the layer loses its stability which is the key property to prevent shear failure, hence rutting or shoving of the surface would be the manifestation. Although, testing was not performed to identify the layer where rutting occurred e.g. trenching, we may postulate with some confidence that the rutting and shoving may be attributed primarily to the layer experiencing stripping.

The stripping in the subbase layer beneath the PCC pavement at Site 5 may also have contributed to the severe faulting at the joints. The low stability would lead to a shoving of the subbase layer, and consequently joint faulting. It is tentatively concluded that the apparent differential deflections observed at the joints with heavy trucks is the primary mechanism causing the faulting. The larger differential deflection is probably due to the lack of load transfer across the joints, since even the good performing rigid pavement at Site no. 1 is experiencing faulting. Thus, lack of load transfer is the primary mechanism with stripping a compounding factor.

CHAPTER 4

LABORATORY TESTING AND RESULTS

On the basis of the diagnostic evaluation of the eight pavement projects in the Oklahoma highway system, a laboratory testing program was established. One objective of this laboratory testing program was to identify material properties or in-place conditions which are likely to be associated with the premature distress of the pavements, including rutting, ravelling, surface wear, cracking etc. The second objective of the experimental program was to provide information for preparing recommendations for modifications to the existing pavement design procedures and pavement management practices.

SAMPLING AND TESTING

In order to perform the laboratory testing, core samples were collected for seven of the eight sites. Core samples for site no. 1 were not collected. The sample collection was performed on two occasions. Table C-1 of Appendix C lists all the core samples collected from the project sites. Sample No. 1 through 35 were collected on June 4-8, 1984 and the remaining samples were collected by the Oklahoma DOT personnel and received at ARE Inc on September 16, 1984. Along with the sample number and site number, Table C-1 gives a brief description of the sample in reference to the depth.

Using the core samples listed in Table C-1, a series of laboratory investigations were performed to determine the physical and mechanical characteristics of the asphalt concrete mix, and to determine the moisture damage potential of the pavements at the project sites. Physical properties determined for the core samples include bulk density, asphalt content, effective specific gravity and air void ratio. The potentials of moisture damage to the asphalt pavement at the projects sites were evaluated by performing the Lottman test (Ref 2). A series of Texas boiling tests (Ref 3) were also performed to determine qualitatively the moisture susceptibility of the asphalt concrete mixtures. The following sections of this chapter describe the procedures followed and the results obtained from these laboratory investigations.

PHYSICAL AND MECHANICAL PROPERTIES

Various physical and mechanical properties of the asphalt concrete mixtures were determined in the laboratory to evaluate the pavement performance of the project sites. These laboratory experimental results are also useful for determining the moisture damage of the pavement. Table C-2 of Appendix C lists the various physical and mechanical properties of the specimens determined in the laboratory and the procedures followed. Results obtained from these experiments are summarized in Table C-3 of Appendix C.

Figure 4.1 shows a plot of bulk density variations between the samples collected from project Site no. 7. This figure shows that there is a large variation in bulk density for all the levels, except for the very lowest level of the core, designated by Level 4. In order to study the uniformity of the asphalt content, a plot was prepared showing the relationships between the bulk density and percent asphalt content. This plot is shown in Figure 4.2 for project Site no. 7. Although a small number of samples were analyzed, the degree of scatter of the points in this figure indicate that the uniformity of the asphalt content in the mixture ranges from fair to good. Similar observations were made for sites 2, 4 and 6 from Figures C-1, C-2 and C-3 of Appendix-C respectively.

Variability of grain size distribution of the mixtures was studied by particle size analysis on the samples from extraction tests and the relationships between bulk density and air voids. The results of sieve analysis for project Site no. 6 for surface materials are shown in Figure 4.3. Similar results of sieve analysis for Sites 2, 4, 6, and 7 are shown in Figures C-4 through C-8 of Appendix-C. These figures show well graded particle size distribution for all the samples except for sample no. 65,

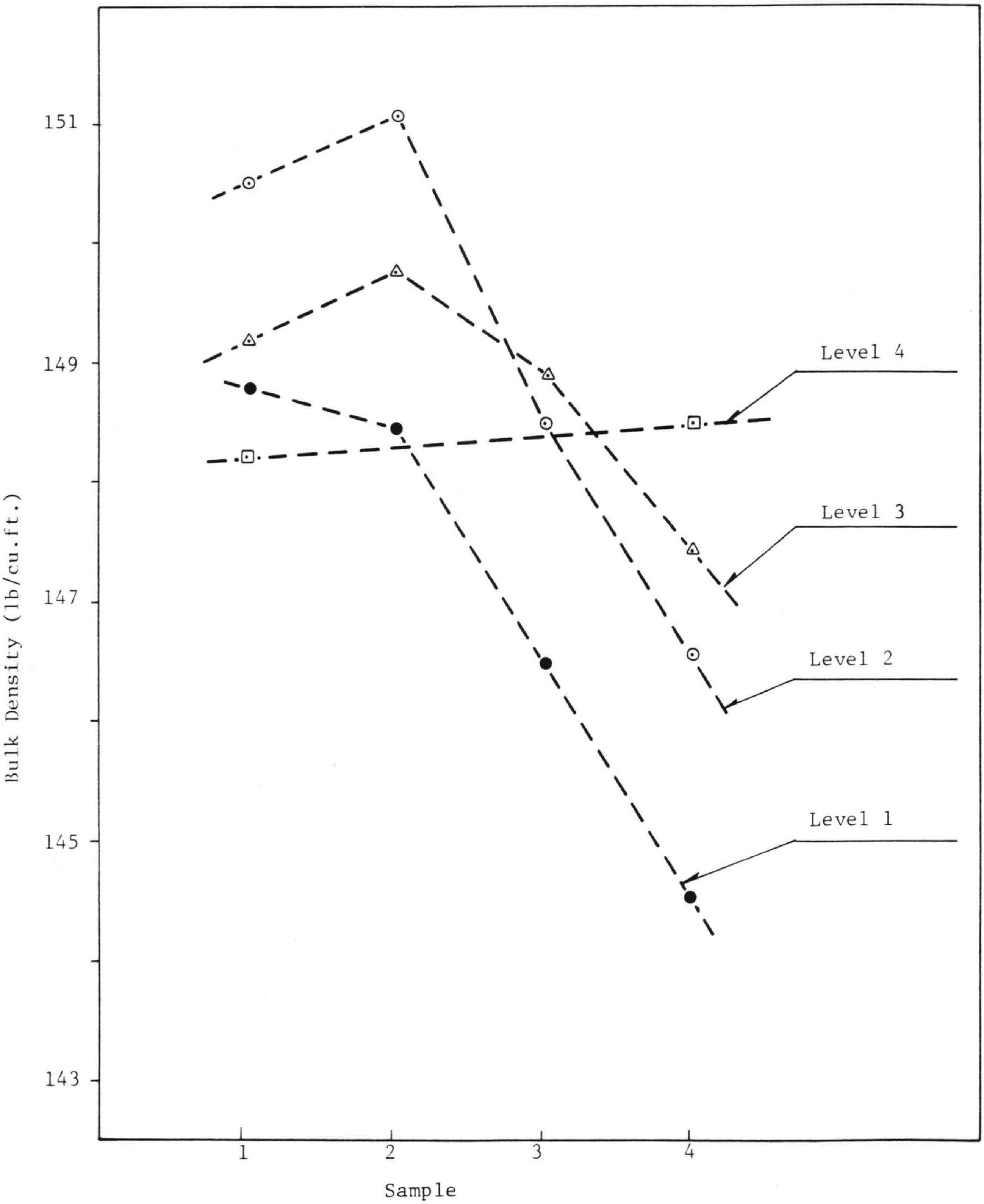


Figure 4.1. Variation of bulk density for site no.7 between different levels of four samples. Level 1 is at the top and Level 4 is at the bottom of the core.

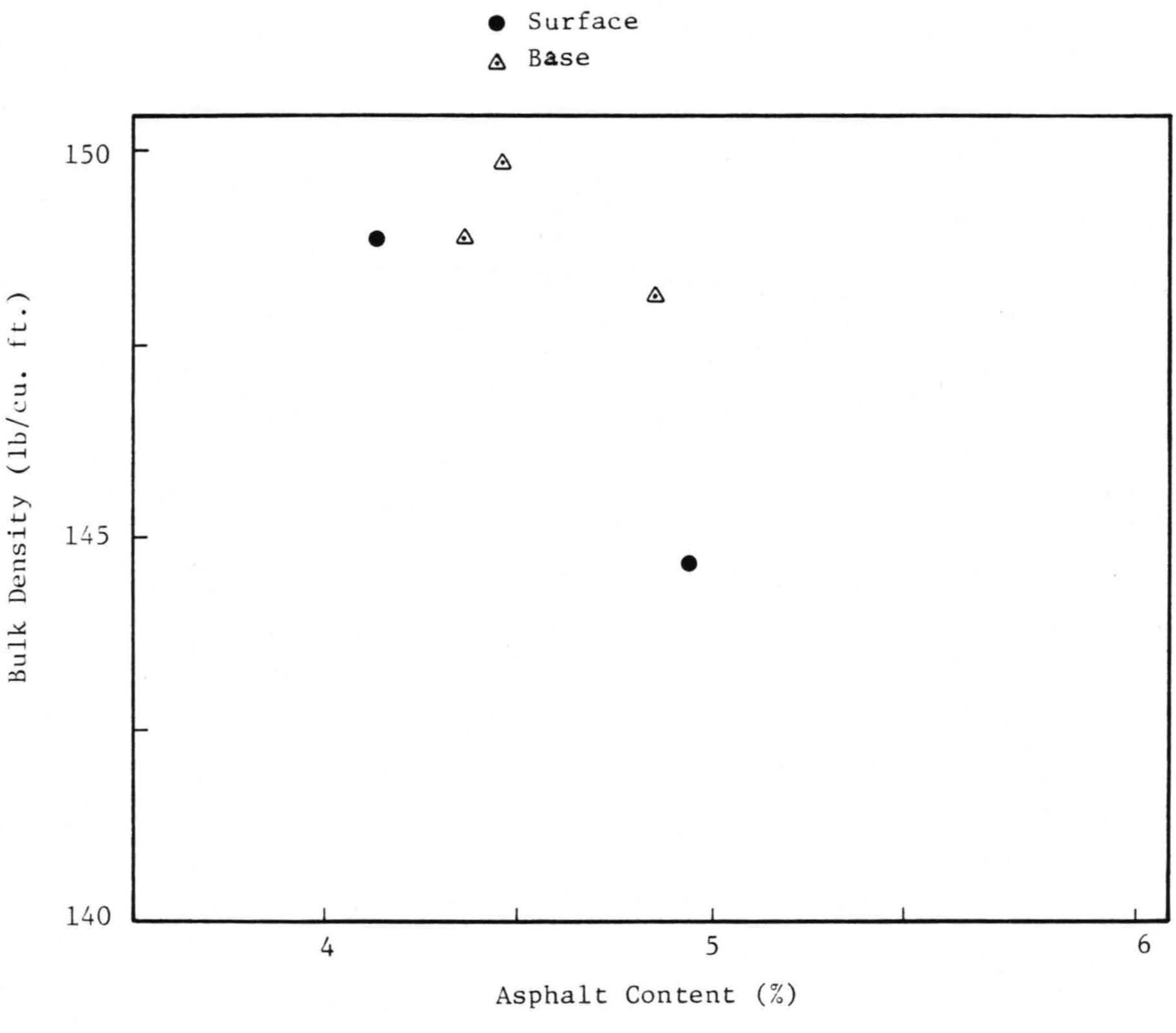


Figure 4.2. Relationships between bulk density and asphalt content for surface and base mixtures of site number 7.

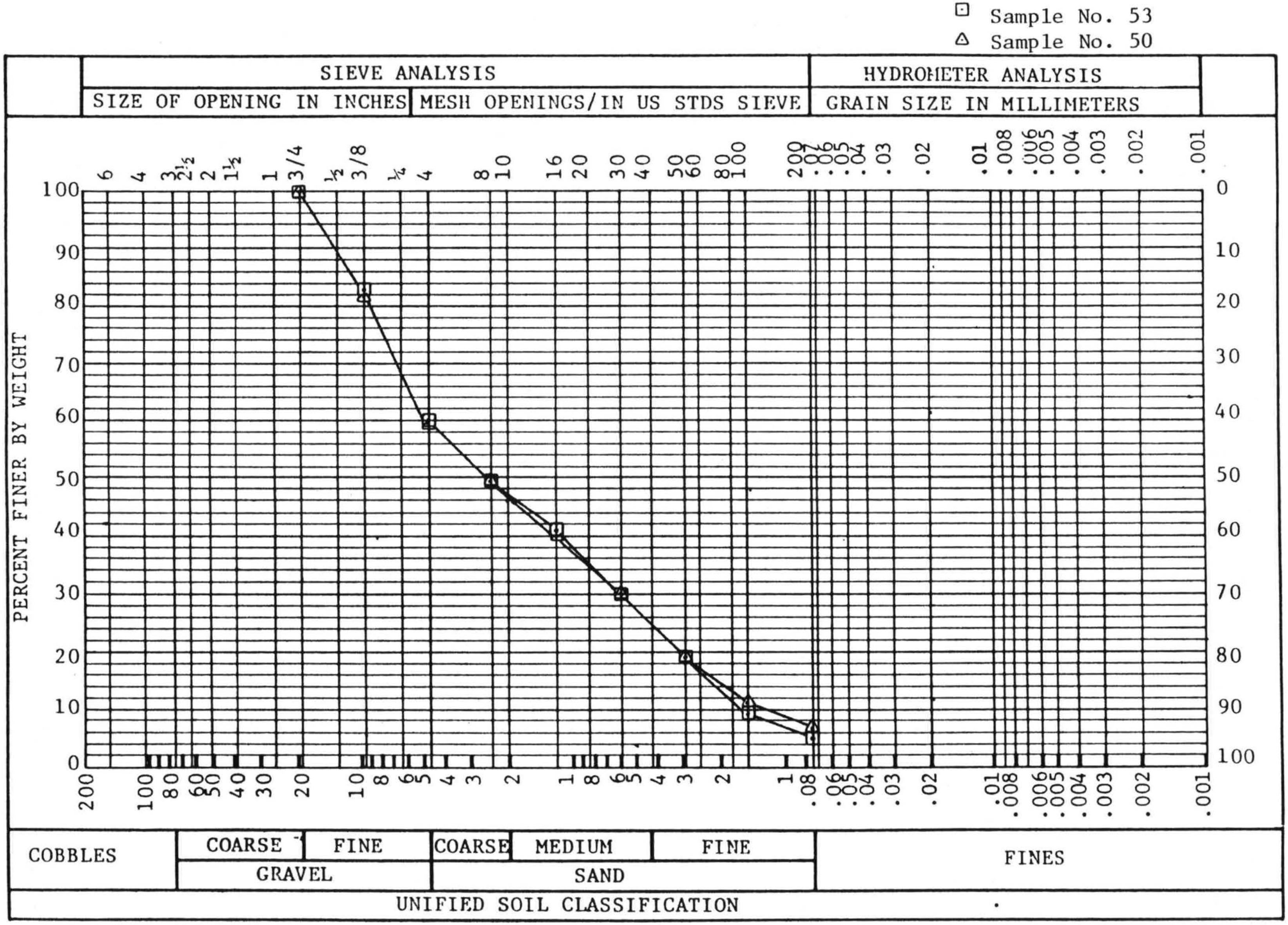


Figure 4.3. Particle size distribution of asphalt concrete surface of Site No. 6.

which represents the surface materials for Site no. 4. For this sample, particles passing sieve no. 200 is over 10 percent, which is quite high. Due to the small number of samples, this finding is not conclusive.

Uniformity of the particle size distribution was also studied by examining the relationships between bulk density and percent air voids. Such a plot for site no. 7 for both surface and base layers are shown in Figure 4.4 Fairly smooth relationships indicates uniform granulometry for both surface and base materials for site no. 7. This figure shows that the percent air voids ranged from 2.5 to 6.2 percent which is considered accpetable for a service asphalt concrete. However, Table C-3 indicates that air voids at some sites exceed 10 percent. The resilient modulus of the specimens for 12 samples from 4 projects sites were determined at 75^{o}F. These values are also summarized in Table C-3. The resilient modulus of the specimens are relatively high, suggesting that the asphalts have aged rapidly or have low temperature susceptibility. It should be noted that the resilient moduli obtained from the field measurements of deflection may not be directly comparable with the laboratory type moduli.

Mechanical properties of the portland cement concrete surface of Sites nos. 5 and 8 were evaluated by performing the ASTM standard test method for splitting tensile strength of cylindrical concrete specimens. The results of these tests are summarized in Table C-4. This table shows that the tensile strengths of both the top and bottom layers of the cylindrical concrete specimens are approximately the same, indicating that the concrete layers have reasonable uniform and high structural strengths.

MOISTURE DAMAGE TESTS

Moisture susceptibility of the asphalt concrete mixture was evaluated by conducting two types of laboratory experiments, namely the Lottman Test and Boiling Test. While the Lottman test determines the moisture susceptibility of the mix by evaluating the mechanical properties of the specimen under different conditions, the boiling test qualitatively

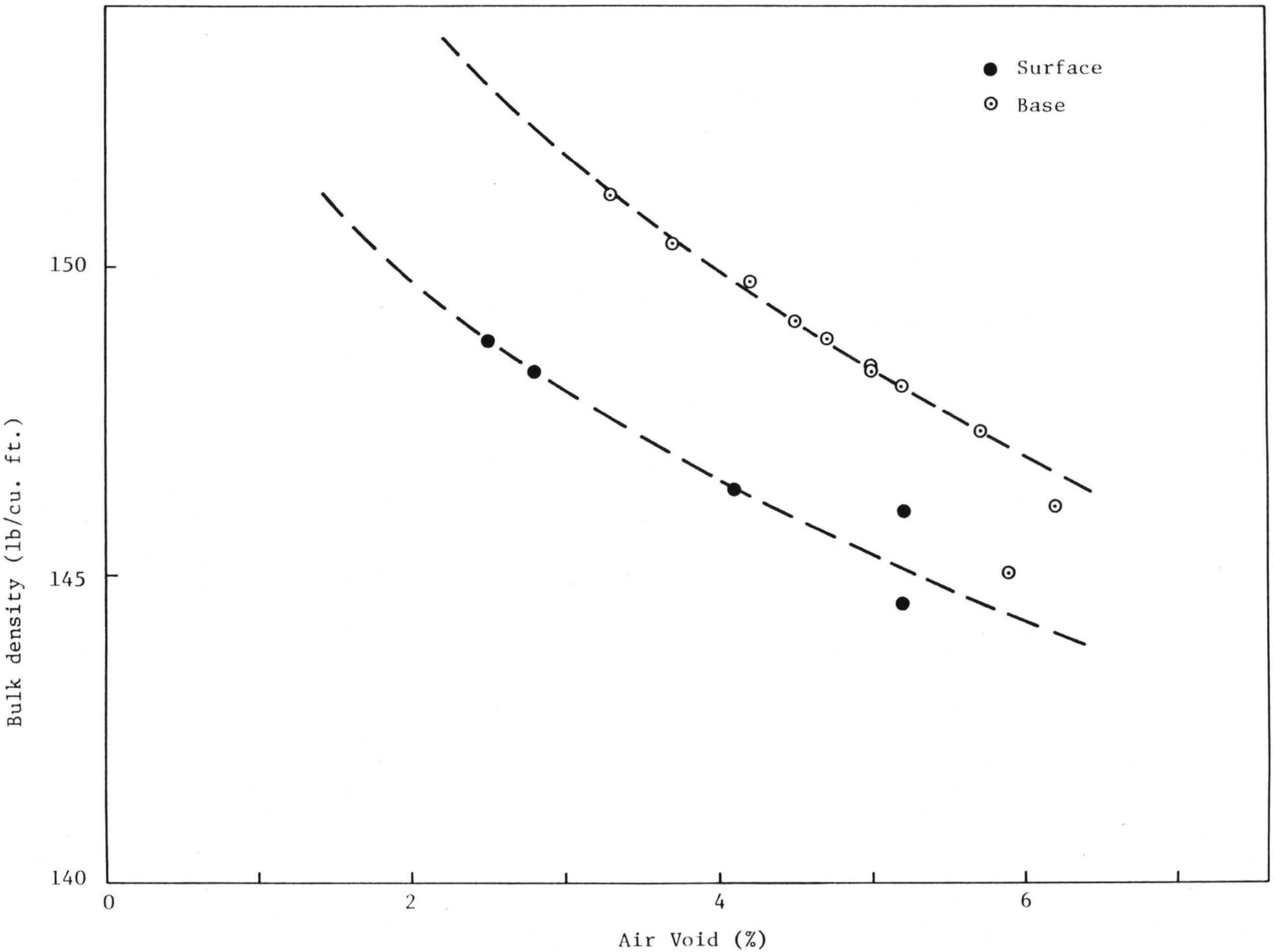


Figure 4.4. Relationships between bulk density and air void for surface and base mixtures of site number 7.

determines the moisture damage by visually estimating the degree of stripping in the specimen after boiling.

Lottman Test

This method is based on the measurement of the diametral tensile strength of the compacted bituminous mixture under accelerated water conditioning and saturation. Internal water pressure in the mixture was produced by vacuum saturation followed by a freeze and warm-water soaking cycle. Comparisons were made between the tensile strength of the dry sample and that of the conditioned sample. A large drop in the diametral tensile strength of the specimen due to the process of conditioning indicates high moisture susceptibility of the asphalt concrete mixture. A detailed description of the Lottman test is available in Reference (2).

The results of the Lottman test are summarized in Table C-5 for the dry specimens and in Table C-6 for the conditioned specimens for different project sites. These tables also give a brief description of the visual observations of the fractured specimen faces after performing the indirect tension test. These visual observations were helpful in this study. Stripping was observed in many instances. However, some specimens did not show any stripping in the visual observation, but indicated moisture damage in the results of the tension tests. Graphical representation of the tensile strength comparisons between dry and conditioned samples along with the bulk density for Site no. 2 are shown in Figure 4.5 and similar graphical representation for other sites are shown in Figures C-9 through C-15 in Appendix-C.

Results from the Lottman test do not confirm the poor moisture susceptibility properties of asphalt concrete based on visual observations of stripping. Since the number of projects and tests involved are relatively small it would be premature to reject the finding from the Lottman tests. It is recommended that an effort should be continued to

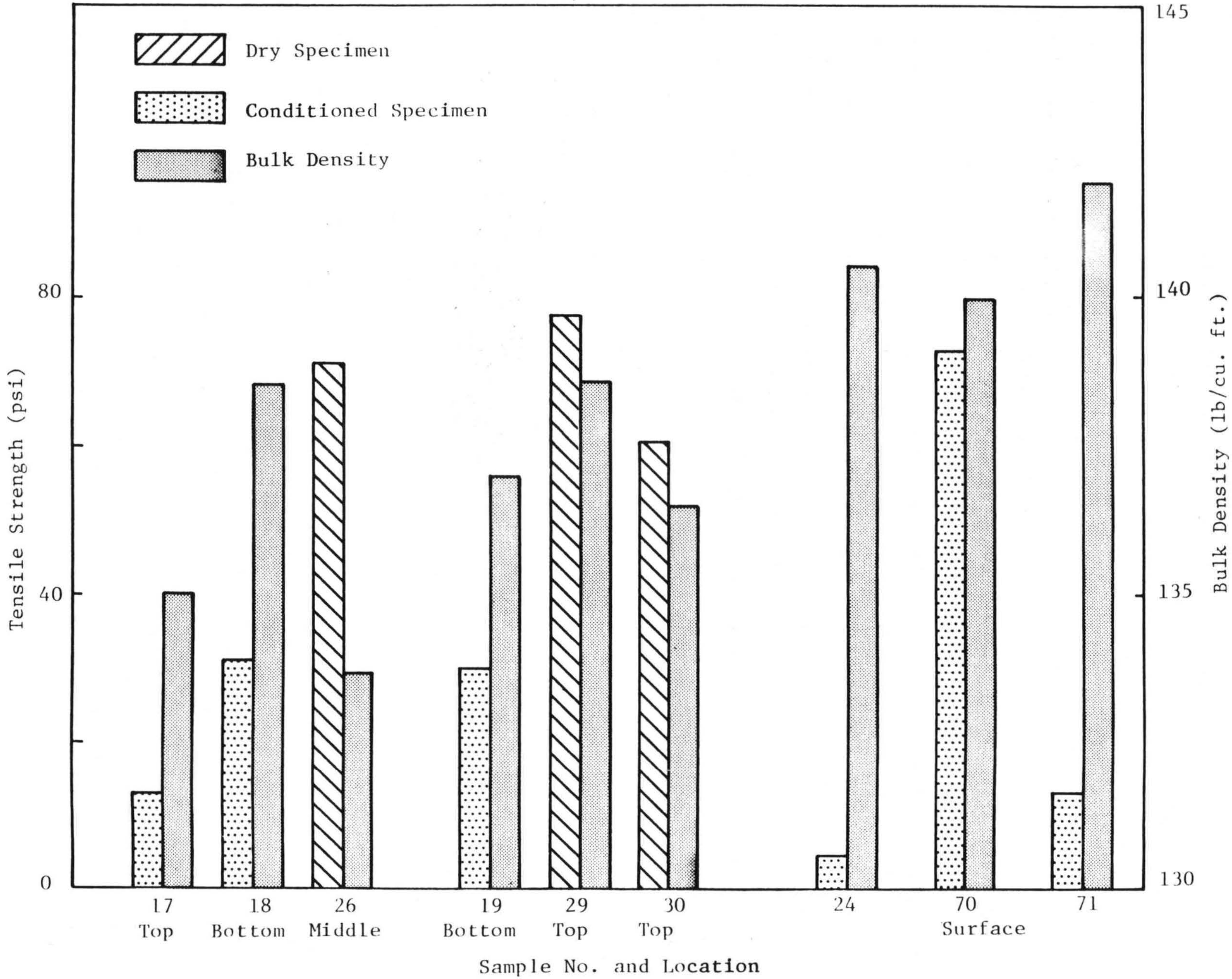


Figure 4.5. Tensile strength for dry and conditioned samples along with bulk density for site no. 2.

determine if the results from the Lottman tests can be correlated with performance.

Boiling Test

This method is used as a screening device for evaluating the moisture susceptibility of an asphalt concrete mixture by visually estimating the degree of stripping after boiling the specimen in distilled water. A detail description of the laboratory experimental procedure of the boiling test is available in reference (3). Results obtained from boiling tests of specimens from different test sites are summarized in Table C-7. The column listing the percentage of asphalt cement retained in Table C-7 indicates the moisture susceptibility of the asphalt concrete mixture. Any value of retained asphalt below 70 percent indicates an unacceptable amount of stripping.

CHAPTER 5

NONDESTRUCTIVE DEFLECTION TESTING AND ANALYSIS

On the basis of diagnostic evaluations of the eight pavement sites in the Oklahoma highway system, a program was established for conducting a series of nondestructive deflection testing (NDT) and analysis. The main objective of this testing program was to determine the Young's moduli of base, subbase, and subgrade for all the eight pavement evaluation sites. The results of these analysis were also used for estimating the pavement's remaining life.

NDT DEVICES

In nondestructive testing of pavements, deflections are measured on the surface as the response of a pavement under test loads. Dynamic force generators in dynamic NDT devices fall into two categories: (1) steady state vibratory force and (2) transient impulse force. In the first category, dynamic deflection is measured as the peak-to-peak amplitude of a deflection signal. In the second case, peak amplitude of a deflection signal is measured as dynamic deflection. The reader is referred to Appendix-D for a detailed explanation of the method and operating characteristics of NDT devices. In this study only the Dynaflect and the Falling Weight Deflectometer (FWD) are considered for NDT evaluation of pavements. The traditional procedure of deflection testing has been the measurement of rebound deflection under a slow moving wheel load, better known as the Benkleman Beam method.

NDT DATA COLLECTION

Nondestructive testing of all 8 test sites was carried out using the Dynaflect and Falling Weight Deflectometer in June 1984. Benkleman Beam deflection data were collected by Oklahoma Department of Highways. The Dynaflect deflection basin is characterized by a set of five deflection measurements at radial distances of 10.0, 15.6, 26.0, 37.4, and 49.0 inches from the center of each loading wheel. The FWD deflection basin is characterized by seven deflection measurements: one at the center of the loading plate and the rest at varying distances from the center of the load plate.

Rigid Pavements

Table 5.1 presents a summary of the number of test locations on rigid pavements, namely Sites no. 1 and no. 5. Deflection basins were measured at a transverse joint and at midslab (between two transverse joints) in the wheel path. Additionally, the Dynaflect deflection basins were also measured in the midspan position, along the center line of the outer lane. Figure 5.1 illustrates the deflection basin measuring scheme.

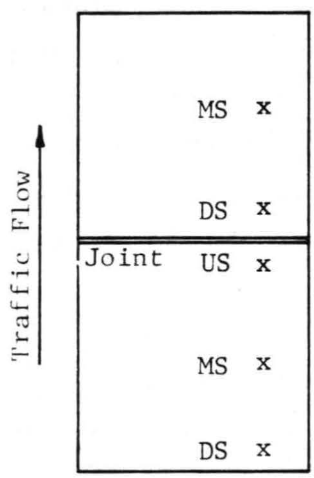
Dynaflect Sensor 5 deflections measured on Site no. 1 (Eastbound) are plotted in Figure 5.2. Research in Texas (Ref 4) has shown that sensor 5 deflection is highly correlated with the elastic modulus of the subgrade. Note that the Sensor 5 deflection is relatively uniform and thus the subgrade is uniform over the test sections. Figure 5.3 illustrates the variation of basin slope with distance. The basin slope reflects the structural condition of the surface layer (Ref 4), i.e. the greater the slope value the lower is the surface stiffness. A listing of the Dynaflect data and similar plots for other sites appears in the compendium of data.

A summary of the FWD data measured on rigid pavements (Sites no. 1 and no. 5) is also included in Table 5.1. At each test location, one or more deflection basins can be measured by the FWD by varying the drop height. Figure 5.4 illustrates a typical set of deflection basins measured at a test location. The deflections are normalized to 1000 lbs by dividing each deflection reading by the corresponding peak-force level. It suggests that these pavements behave as a linear system within this load range as shown in Figure 5.5 where Sensor 1 and Sensor 7 deflections

Table 5.1. Summary of deflection basins measured on flexible and rigid pavement site.

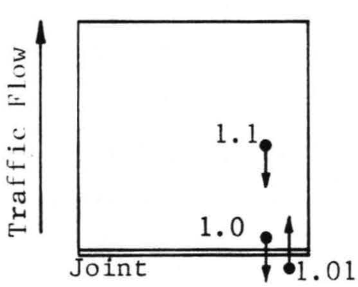
Pavement Type	Site No.	Traffic Direction	Dynaflect	FWD			
				5000	9000	11000	16000
Rigid	1	E	15	10	10	--	--
		W	15	10	10	--	--
	5	N	12	--	2x12	--	12
		S	12	--	2x11	--	11
Flexible	2	N	26	13	2x13	13	--
		S	25	13	2x13	13	--
	3	E	36	--	2x18	--	18
		W	47	--	2x24	--	24
	4	S	65	33	2x33	33	--
	6	N	8	--	2x10	--	10
		S	--	--	2x 9	--	9
	7	N	14	--	2x 7	--	7
		S	14	--	2x 7	--	7

(a) DYNAFLECT



Letter Code	Direction	Position
1	NB	MS (Midspan)
2	SB	
3	EB	
4	WB	
5	NB	US (Before joint)
6	SB	
7	EB	
8	WB	
A	NB	DS (After joint)
B	SB	
C	EB	
D	WB	

(b) FWD



Letter Code	Position
1.1	Midspan
1.0	At joint
1.01	(Extra tests on joints at site #5)

Figure 5.1. Deflection basin measurements on rigid pavements.

OKLAHOMA PAVEMENT EVALUATION
DYNAFLECT MEASUREMENTS

DATE	: 06/84	PROJECT NO	: TOK-1
PAVEMENT ID	: I-40 EB (SITE 1) P	CLIENT	: ODOT
LOCATION	: OKLAHOMA		

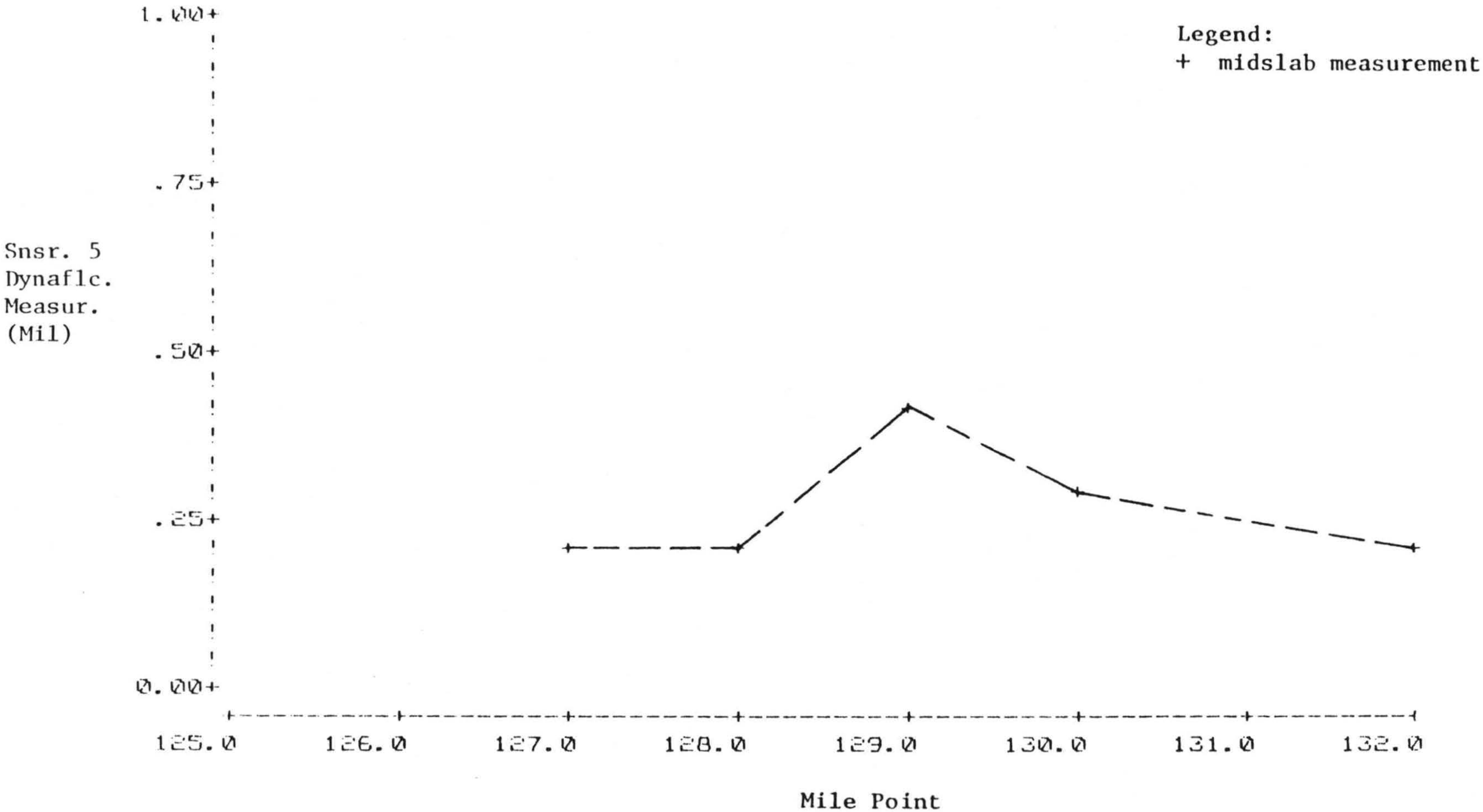


Figure 5.2. Variation of sensor 5 Dynaflect deflections with distance

OKLAHOMA PAVEMENT EVALUATION
DYNAFLECT MEASUREMENTS

DATE	: 06/84	PROJECT NO	: TOK-1
PAVEMENT ID	: I-40 EB (SITE 1) P	CLIENT	: ODOT
LOCATION	: OKLAHOMA		

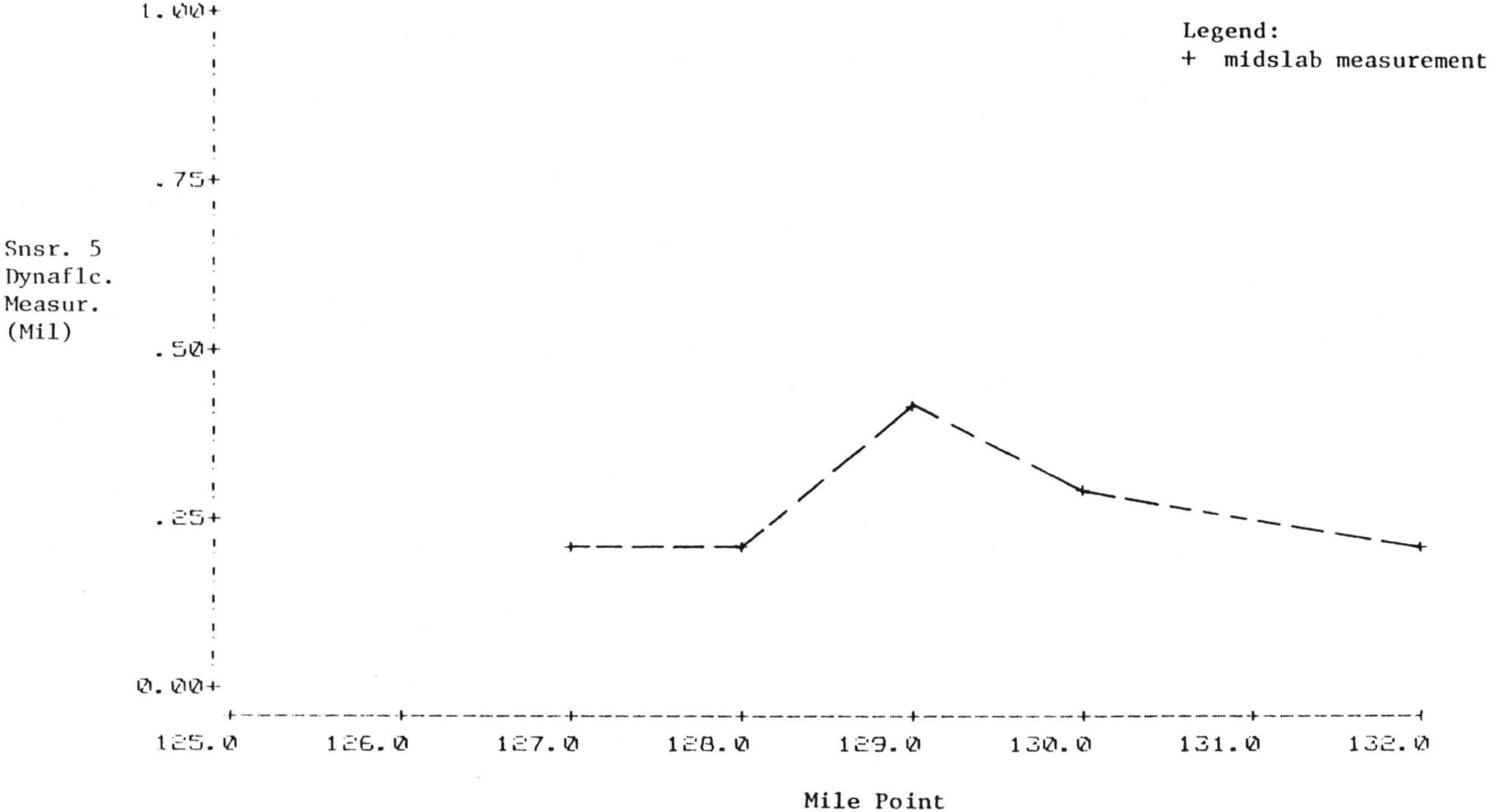


Figure 5.2. Variation of sensor 5 Dynaflect deflections with distance

OKLAHOMA PAVEMENT EVALUATION

DYNAFLECT MEASUREMENTS

DATE	: 06/84	PROJECT NO	: TOK-1
PAVEMENT ID	: I-40 EB (SITE 1) P	CLIENT	: ODOT
LOCATION	: OKLAHOMA		

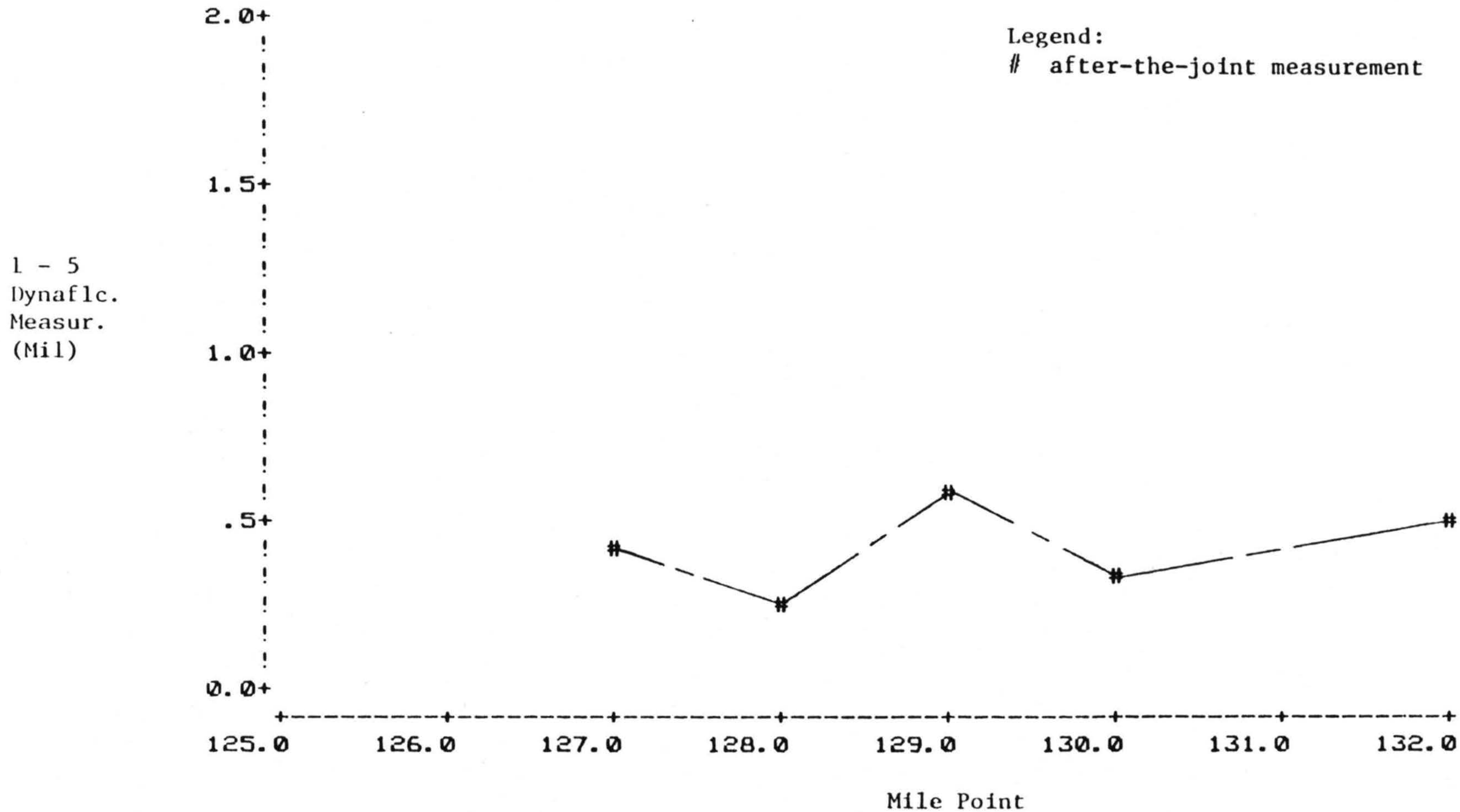
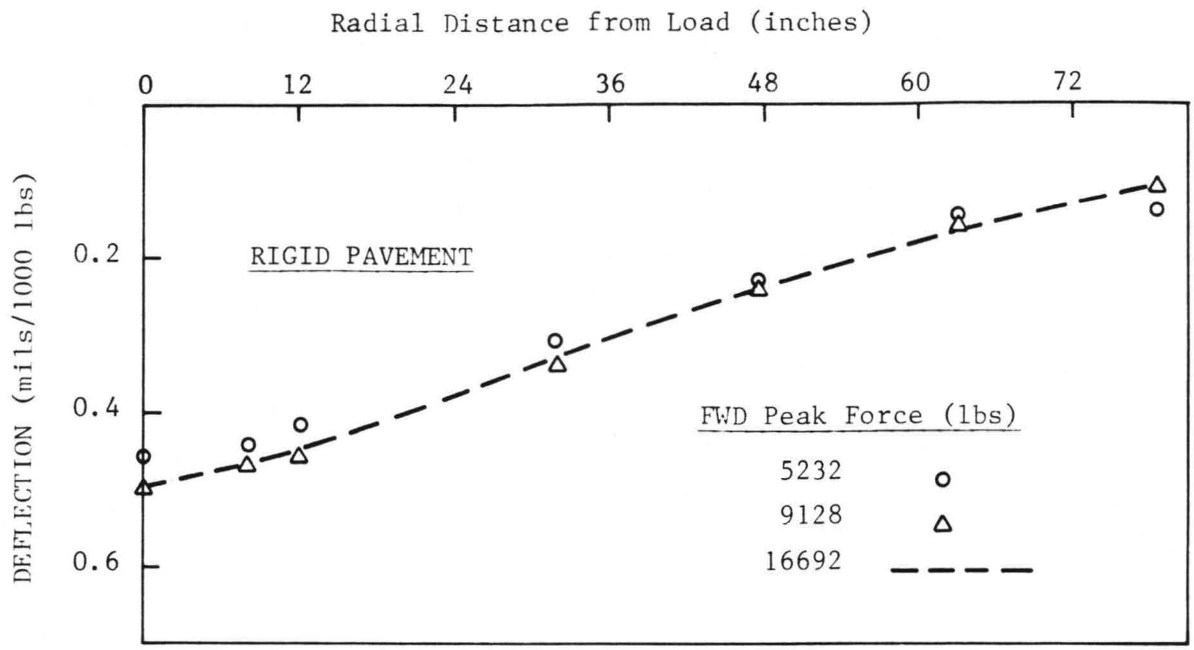
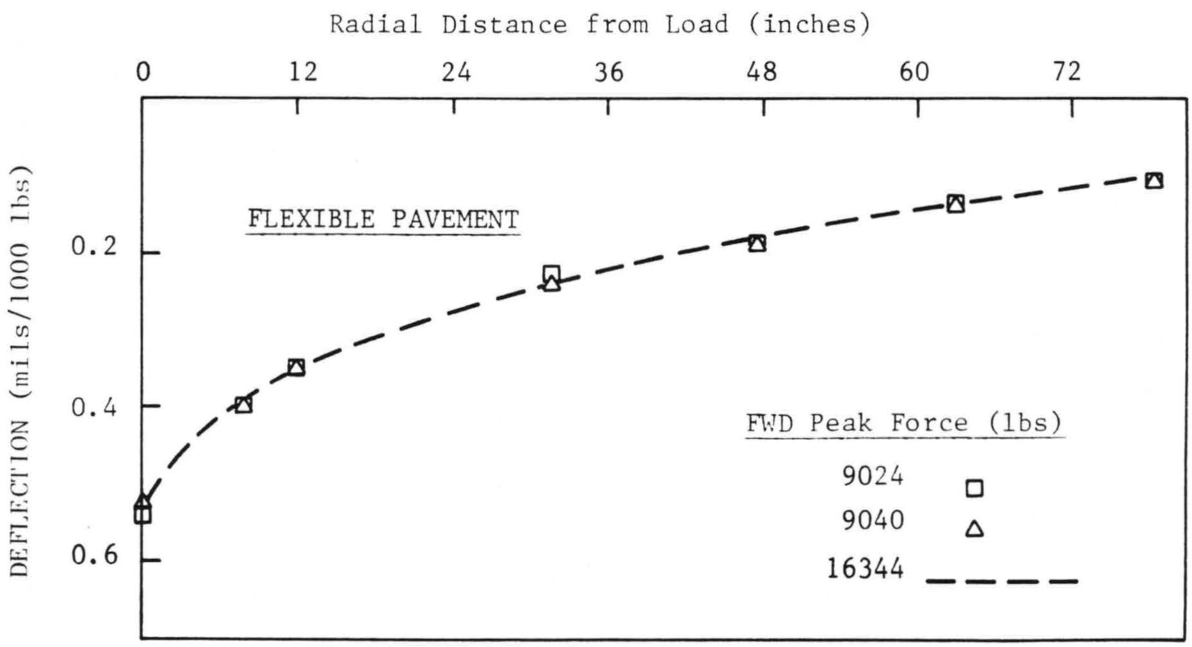


Figure 5.3. Variation of Dynaflect deflection parameter, SLOPE (sensor 1 - sensor 5) with distance.



(a) SITE #1, WB (FILE: I40W2, RCD: 18, STN: 132.1)



(b) SITE #6, NB (FILE: I35N5, RCD: 18, STN: 4.5)

Figure 5.4. Typical FWD deflection basins at different peak force levels.

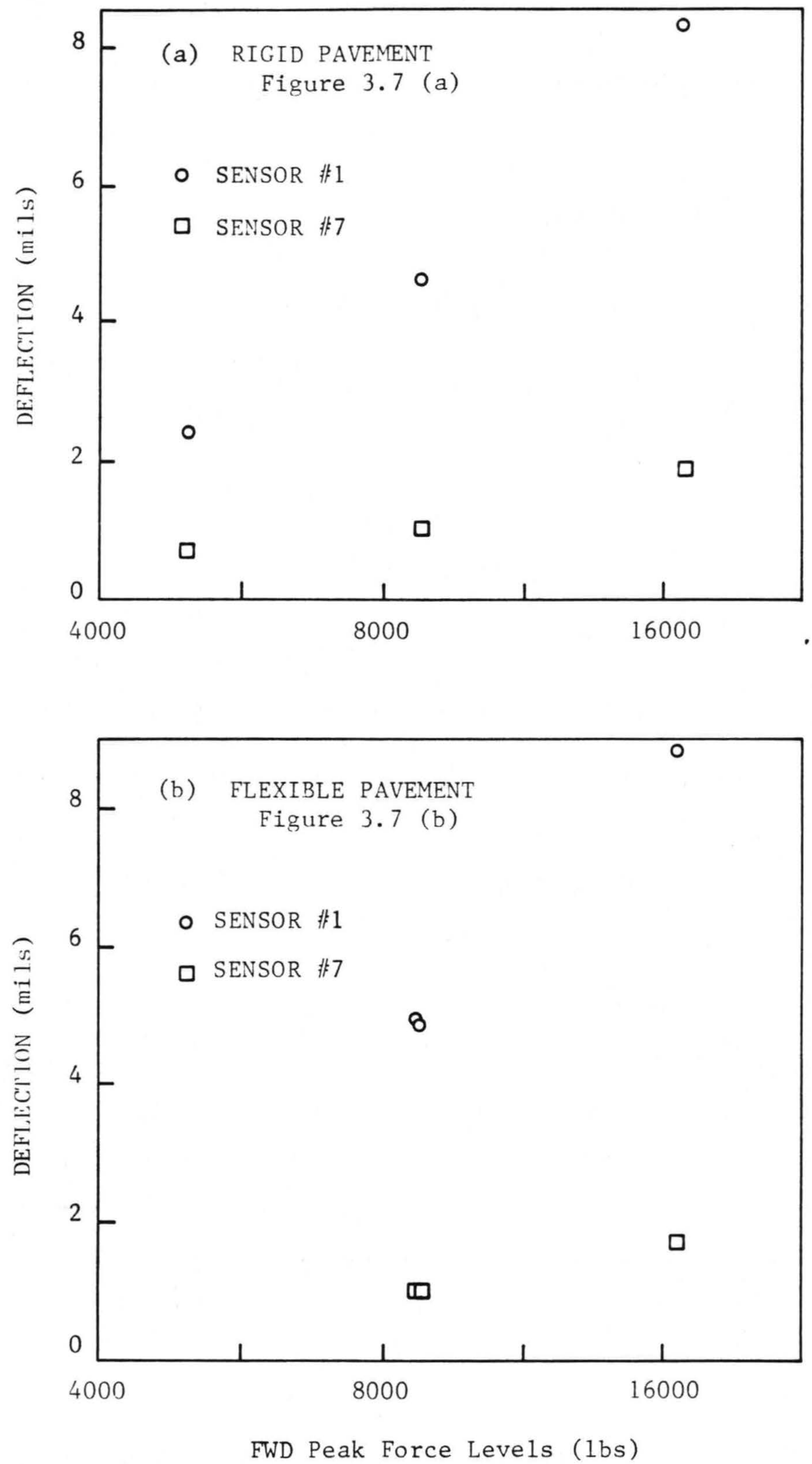


Figure 5.5. Deflection verses FWD peak force level relationship at sensors #1 and #7.

have been plotted versus FWD peak-force amplitudes. A listing of the FWD and Benkleman Beam deflection data are presented in the compendium of data.

Flexible Pavements

Table 5.1 also summarizes the NDT data collected on flexible pavements (Sites no. 2, 3, 4, 6, and 7). Dynaflect deflection basins were collected in the wheelpath as well as on the centerline of the outside lane. The Dynaflect, FWD and Benkleman Beam deflection data are presented in the compendium of data. FWD deflection basins were measured in the wheelpath. To examine the influence of peak-force levels on measured deflections, Figures 5.4 and 5.5 have been prepared. It can be concluded that FWD force levels used in the measurements are within the linear range.

Composite Pavements

Site no. 8 is an example of a composite pavement where an existing rigid pavement was overlaid with a layer of asphaltic concrete mix. The Dynaflect, FWD, and Benkleman Beam deflection data are shown in the compendium of information in a separate volume.

DATA RELATED TO PAVEMENT LAYERS

The analysis of the deflection data was accomplished by using the latest state-of-the-art, and the reader is referred to Appendix-E for a detailed description of the procedure. For the analysis of deflection basin data, it is necessary to have known values of thickness of each pavement layer, Poisson's ratio and the type of material used in each layer. For this purpose, construction plans and pavement design sheets provided by the Oklahoma Department of Highways were thoroughly studied. Figure 5.6 illustrates pavement structures pertaining to each test site based on the design information. The same figure also illustrates the

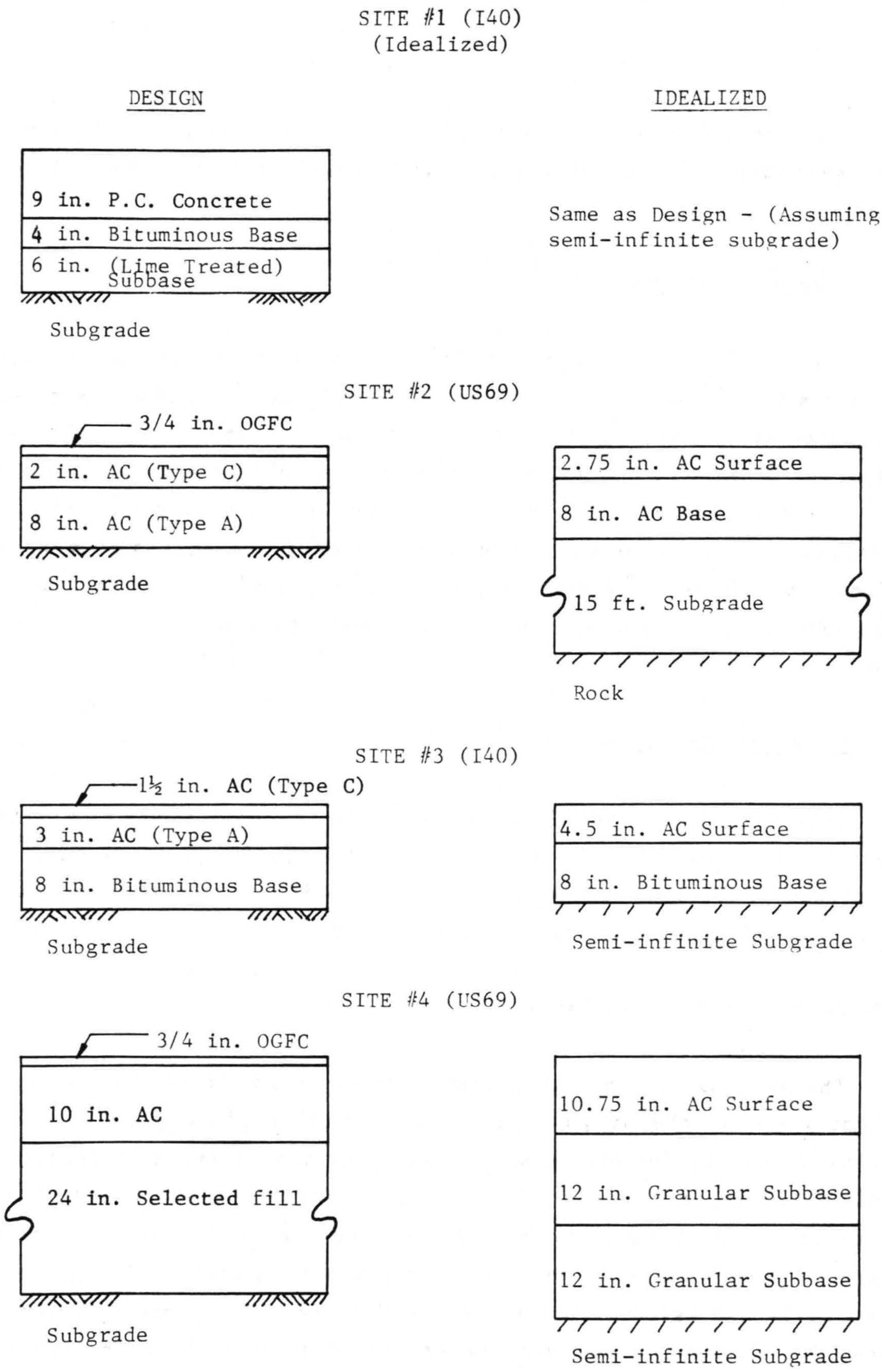


Figure 5.6. Idealized pavement structures assumed for basin fitting and structural response analysis.

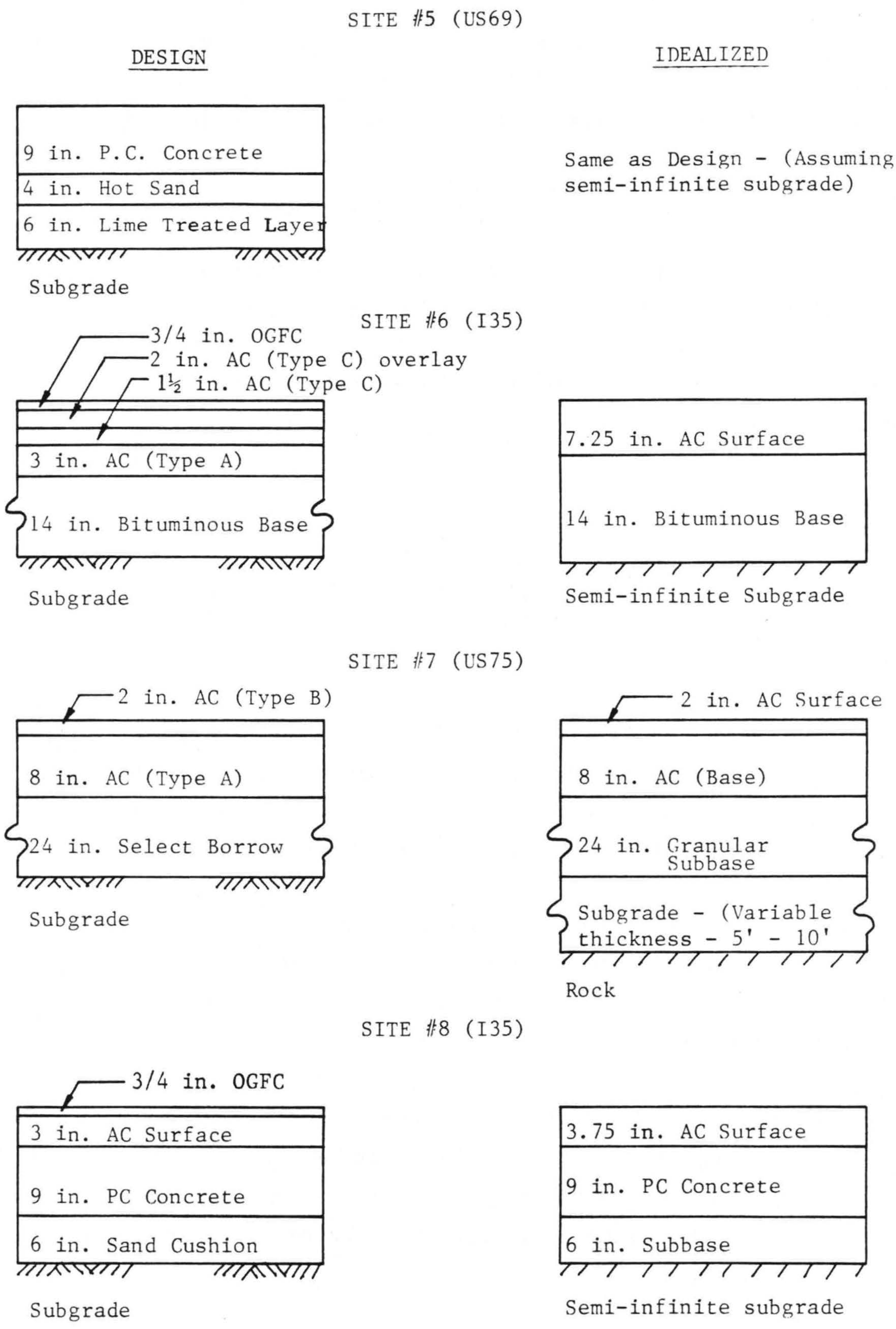


Figure 5.6. (Continued)

representative pavement structures assumed for evaluating the measured deflection basins. Table 5.2 summarizes Poisson's ratios of typical pavement materials assumed for the analysis presented in this study.

SELECTION OF DEFLECTION DATA FOR ANALYSIS

For the structural evaluation of pavements at the test sites, the following guidelines were used to select the deflection basins from the raw data for analysis. These analysis procedures are based on the discussion presented in Appendix-E. From this analysis the properties of each layer are obtained. These properties are compared between good and failed sections, and are also used for remaining life analysis presented later in this chapter.

(1) For rigid pavements (Sites no. 1 and no. 5) the deflection basins measured in the midspan position (centerline of lane) are to be evaluated.

(2) Deflection basins measured in the wheelpath were analyzed for the evaluation of flexible pavements (Sites nos. 2, 3, 4, 6, and 7). A comparison of wheel path and between the wheel path deflections provide an indication of the load effect.

(3) In the case of the composite pavement (Site no. 8), basins measured at the centerline are preferred for use in the analysis. However, FWD data were available only in wheelpath locations.

(4) At each test location, 3 or 4 deflection basins were recorded during FWD tests at varying drop heights. For this study, FWD deflection basins measured at the second drop height (corresponding to a peak-force level of around 9000 lbs) are evaluated.

Table 5.2. Poisson's ratios of different pavement materials assumed in this study.

Material	Poisson's Ratio
P.C. Concrete	0.15
Asphaltic concrete surface course	0.30
Bituminous base course	0.35
Subbase	0.40
Selected fill	0.45
Subgrade	0.45

The deflection data are also used directly to estimate the load transfer efficiency across the transverse joint. A ratio of the joint deflection to the mid-span deflection produces an indicator of load transfer, i.e. the greater the ratio the less the load transfer.

The representative pavement structures for the test sites assumed in these analyses are presented in Figure 5.6. The results of these analyses are presented in the following sections.

IN SITU MATERIAL CHARACTERIZATION

The detailed outputs from RPEDD1 and FPEDD1 described in Appendix-E contain the results of each individual iteration, a summary of the best iteration with the least discrepancy in the measured and computed deflections, strain sensitive moduli of granular layers and subgrade, (including temperature-corrected asphaltic concrete modulus in the case of FPEDD1) and the remaining life. Finally, the results are summarized in the outputs generated by these programs. The results presented in this study are based on the final tabulated results from these computer programs. Poisson's ratios assumed for different pavement materials are presented in Table 5.2. The results of the Dynaflect deflection basins include corrected moduli of nonlinear strain-sensitive layers. A detailed description of the structural evaluation methodology used in this study is contained in Reference 5.

<u>Rigid Pavements</u>

The deflection basins measured on Site no. 1 (eastbound and westbound) and no. 5 (northbound and southbound) were analyzed using program RPEDD1. Tables 5.3 (a) and (b) present in situ Young's moduli evaluated from the Dynaflect and FWD deflection basins measured at Site no. 1 (IH 40 eastbound, EB). Figure 5.7 illustrates the variation of the modulus of each layer with distance. Note the relative constant values of the subgrade and variability in the other layers. Similar results for the

Table 5.3. In situ Young's moduli determined from deflection basins measured on Site #1. (IH-40, EB)

(a) Dynaflect

Station		Final Values of Youngs Moduli (PSI)			
		PC Concrete	AC Base	Subbase	Subgrade
1	127	3,397,000	307,000	154,000	26,400
2	128	3,444,000	205,000	182,000	24,900
3	129.01	3,306,000	350,000	151,000	12,400
4	130	3,503,000	100,000	199,000	17,600
5	132	4,070,000	205,000	34,000	24,200
Mean:		3,544,000	233,000	144,000	21,100
Std. Dev:		302,900	97,900	64,600	5,930
C.V.,%:		8.6	42.0	44.9	28.1

(b) FWD

Station		Final Values of Youngs Moduli (PSI)			
		PC Concrete	AC Base	Subbase	Subgrade
1	127	3,973,000	268,000	57,000	32,200
2	128	4,293,000	267,000	91,400	29,500
3	129	4,542,000	205,000	159,000	21,800
4	130	3,417,000	118,000	54,900	22,900
5	132	3,481,000	138,000	32,400	31,000
Mean:		3,941,000	199,000	78,900	27,500
Std. Dev:		492,900	70,200	49,300	4,810
C.V.,%:		12.5	35.2	62.5	17.5

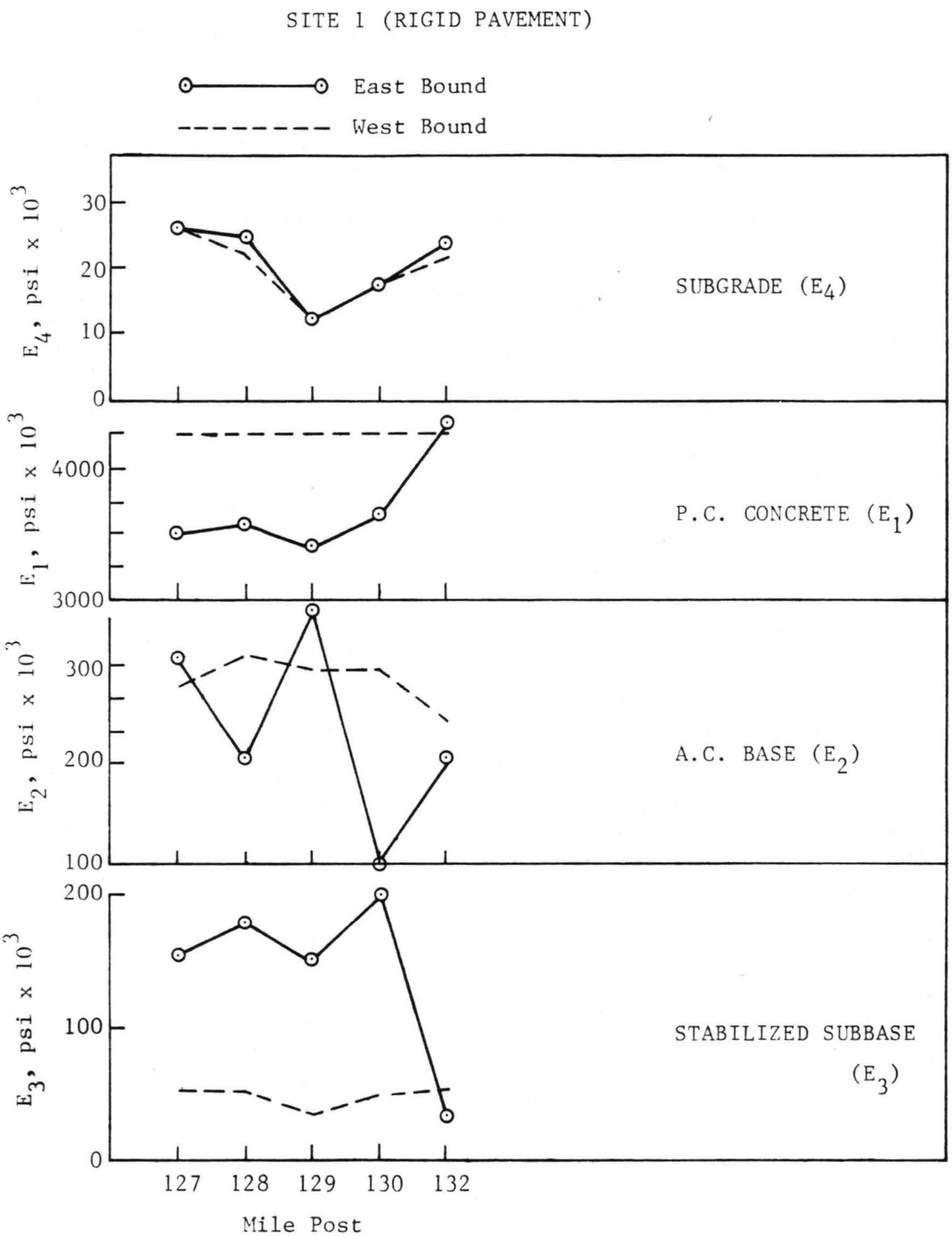


Figure 5.7. Variation of in situ Young's moduli (determined from the analysis of Dynaflect deflection basins) with distance.

other rigid pavement sites are given in Table F-1, F-2, and F-3 of Appendix F.

Flexible Pavements

The Site no. 2 (NB) in situ Young's moduli of pavement layers evaluated from deflection basins are summarized in Tables 5.4 (a) and (b) for the Dynaflect and FWD respectively. Similar results for other sites are presented in Appendix F in Table F-4 through F-10.

It is noted that Site no. 7 has a rock formation at a shallow depth varying from 5 ft to 10 ft. It is apparent from the low Dynaflect deflections at sensor 5 and was confirmed by the subsoil record at this site. If the subgrade is assumed semi-infinite, then the modulus would be over estimated. Therefore, it is important to enter the actual thickness of subgrade modulus. The computer program, FPEDD1 (Ref 5), in this case calls a special subroutine to predict the seed modulus of the subgrade with consideration to the influence of a rock layer on surface deflections.

Composite Pavement

Site no. 8 is a rigid pavement overlaid with an asphaltic concrete layer. Program FPEDD1 was used to analyze deflection basins measured on this site. The estimated seed modulus of concrete layer was entered in the inputs and this layer was specified as a stabilized layer. The Site no. 8 (NB) Young's moduli of pavement layers evaluated from deflection basins are summarized in Table 5.5 (a) and (b) for Dynaflect and FWD respectively. Similar results for SB lanes are shown in Table F-11 of Appendix F.

Table 5.4. In situ Young's moduli at Site #2 (US-69, NB)

(a) Dynaflect

Station		Final Values of Youngs Moduli (PSI) A.C. Surface Layer, (E_1)	E_1*	A.C. Base	Subgrade
1	4.54	741,000	850,000	800,000	28,000
2	4.40	700,000	700,000	289,000	15,600
3	4.20	645,000	700,000	150,000	11,300
4	4.00	700,000	700,000	312,000	17,000
5	3.80	338,000	454,000	74,600	43,200
6	3.60	134,000	180,000	53,700	19,200
7	2.40	144,000	194,000	70,000	48,100
8	2.20	379,000	509,000	325,000	17,300
9	2.00	444,000	597,000	374,000	28,100
10	1.80	246,000	331,000	72,500	42,900
11	1.60	181,000	243,000	51,400	26,700
12	1.40	162,000	218,000	49,300	26,600
13	1.20	177,000	238,000	48,800	16,800
Mean:		384,000	455,000	205,000	26,200
Std. Dev:		237,000	236,000	217,000	11,900
C.V.%:		61.7	51.8	106	45.2

(b) FWD

Station		Final Values of Youngs Moduli (PSI) A.C. Surface Layer, (E_1)	E_1*	A.C. Base	Subgrade
1	4.54	224,000	301,000	439,000	19,000
2	4.40	266,000	357,000	465,000	15,500
3	4.20	228,000	306,000	269,000	16,500
4	4.00	327,000	440,000	362,000	15,900
5	3.80	171,000	230,000	137,000	35,600
6	3.60	20,000	26,900	123,000	22,000
7	2.40	74,400	100,000	238,000	68,000
8	2.20	20,000	26,900	116,000	33,100
9	2.00	132,000	177,000	98,500	28,100
10	1.80	86,900	117,000	180,000	51,400
11	1.60	142,000	191,000	104,000	23,000
12	1.40	97,200	131,000	97,100	22,800
13	1.20	20,000	26,900	164,000	16,900
Mean:		139,000	187,000	215,000	28,700
Std. Dev:		99,600	134,000	131,000	15,600
C.V.%:		71.5	71.5	60.9	54.5

*E_1 corrected for design temperature

Table 5.5. In situ Young's moduli at Site #8 (I-35, NB)

(a) Dynaflect

Station		Final Values of Youngs Moduli (PSI)				
		E_1	E_1*	P.C.C.	Subbase	Subgrade
1	0	180,000	538,000	1,500,000	11,200	28,100
2	1.00	180,000	538,000	1,500,000	11,200	28,900
3	2.00	277,000	700,000	2,229,000	5,200	18,900
4	3.00	235,000	700,000	1,916,000	2,400	20,900
5	4.00	180,000	538,000	1,500,000	11,200	29,700
6	5.00	180,000	538,000	1,500,000	11,200	18,600
7	6.00	180,000	538,000	1,500,000	13,100	16,200
8	7.00	180,000	538,000	1,500,000	11,200	25,400
9	8.00	180,000	538,000	1,500,000	11,100	37,800
10	9.00	180,000	538,000	1,500,000	11,200	25,000
11	10.00	180,000	538,000	1,500,000	11,100	40,800
12	11.00	219,000	655,000	1,813,000	13,300	18,000
13	11.00	207,000	618,000	1,500,000	11,200	19,000
Mean:		197,000	578,000	1,612,000	10,400	25,100
Std.Dev:		30,300	658,000	230,000	3,060	7,780
C.V.,%:		15.4	11.4	14.3	29.5	30.9

(b) FWD

Station		Final Values of Youngs Moduli (PSI)				
		E_1	E_1*	P.C.C.	Subbase	Subgrade
1	11.0	485,000	700,000	4,000,000	16,500	23,800
2	10.0	268,000	700,000	1,500,000	35,500	50,100
3	9.0	205,000	613,000	1,918,000	72,300	24,100
4	8.0	180,000	538,000	1,500,000	21,100	48,900
5	7.0	180,000	538,000	1,500,000	82,500	27,500
6	6.0	180,000	538,000	1,500,000	61,400	20,500
7	5.0	180,000	538,000	1,500,000	67,700	22,600
8	4.0	180,000	538,000	1,500,000	60,400	27,400
9	3.0	180,000	538,000	2,198,000	72,800	24,500
10	2.0	393,000	700,000	3,407,000	44,500	20,900
11	1.0	314,000	700,000	2,341,000	61,800	28,100
12	0	250,000	700,000	1,718,000	49,400	26,200
Mean:		250,000	612,000	2,048,000	53,800	28,700
Std. Dev:		101,000	81,000	836,100	20,900	10,000
C.V.,%:		40.4	13.2	40.8	38.8	34.9

*E_1 corrected for design temperature

STRUCTURE RESPONSE ANALYSIS

Critical structural response analysis has been performed for the deflection basin at each test site as a part of structural evaluation using programs RPEDD1 for rigid pavements, and FPEDD1 for flexible pavements (See Appendix E). For remaining life computations, very rough estimates of past 18 kips equivalent single axle load applications based on design data were used (Table 5.6).

Results of the structural response analysis for rigid pavement of Site no. 1, Eastbound are presented in Table 5.7. Similar results from other rigid pavement sites are shown in Table F-12, F-13, and F-14 of Appendix F. Results for flexible pavement sites are shown in Tables F-15 through F-23 of Appendix F. A summary of the mean remaining life estimates for both rigid and flexible pavement sites is shown in Table 5.8. In the case of Site 8 (composite pavement); results of the structural response analysis for the Dynaflect and the FWD are presented in tables F-25 and F-26 respectively in Appendix F. In all these tables, a value of zero in the horizontal critical response at the bottom of surface layer indicates that the response is compressive resulting in unlimited fatigue life (or 100 percent remaining life).

DISCUSSION

A general discussion of the results included presented in this chapter on the basis of the mechanistic evaluation of dynamic deflection basins is presented in this section; including a comparison of the results evaluated from the analyses of the Dynaflect and the FWD.

Rigid Pavements

A summary of average results for Sites no. 1 and 5 are presented in Table 5.9. The analyses of both the Dynaflect and FWD deflection basins at Site no. 1 (east and west bound) have generated Young's moduli of

Table 5.6. Summary of design traffic data.

Site	Pavt. Type	Year (constructed)	Heavy Commercial Traffic[1]	Years In-service	Approx. 18-kips ESAL
1	R[2]	1965	1348	19	6,888,000
2	F[3]	1977	1204	7	3,077,000
3	F	1975	800	9	2,628,000
4	F	1980	936	4	1,366,000
5	R	1977	1821	7	4,653,000
6	F	1975	1044	9	3,429,000
7	F	1978	267	6	585,000
8	Overlaid	1978	___	___	___

[1] Based on average ADT and % heavy commercial traffic recorded from the design report of each site.

[2] Rigid Pavement.

[3] Flexible Pavement.

Table 5.7. Structural response and remaining life analyses, Site #1 (I-40, EB).

(a) Dynaflect

Station		Max. Deflection (Mils)	Tensile Stress (psi)	Deviator Stress (psi)	Remaining Life (%)
1	127	3.7	68.2	-1.73	55.0
2	128	3.9	73.1	-1.75	44.5
3	129.01	6.3	71.5	-1.18	48.2
4	130	5.0	84.5	-1.61	14.4
5	182	4.1	91.3	-1.77	0.0
				Mean:	32.4
				Std.Dev:	23.9
				C.V.%:	73.7

(b) FWD

Station		Max. Deflection (Mils)	Tensile Stress (psi)	Deviator Stress (psi)	Remaining Life (%)
1	127	3.3	81.0	-1.98	24.6
2	128	3.4	81.1	-1.76	24.1
3	129	4.0	84.5	-1.48	14.3
4	130	4.5	90.0	-1.86	0.0
5	132	3.7	88.5	-2.20	1.4
				Mean:	12.9
				Std.Dev:	11.9
				C.V.%:	92.0

Table 5.8. Summary of Mean Remaining Life Estimates.

Pavement Type	Site No.	Traffic Direction	Estimated Remaining Life (%) Dynaflect	FWD
Rigid	1	Eastbound	32.4	12.9
		Westbound	9.1	64.4
	5	Northbound	49.3	36.9
		Southbound	38.7	31.5
Flexible	2	Northbound	46.7	99.3
		Southbound	99.3	90.0
	3	Eastbound	49.2	43.5
		Westbound	84.8	86.0
	4	Southbound	30.6	36.1
	6	Northbound	99.9	100.0
		Southbound	----	99.9
	7	Northbound	99.8	100.0
		Southbound	99.7	100.0

Table 5.9. Summary of average in situ moduli for rigid pavement sites.

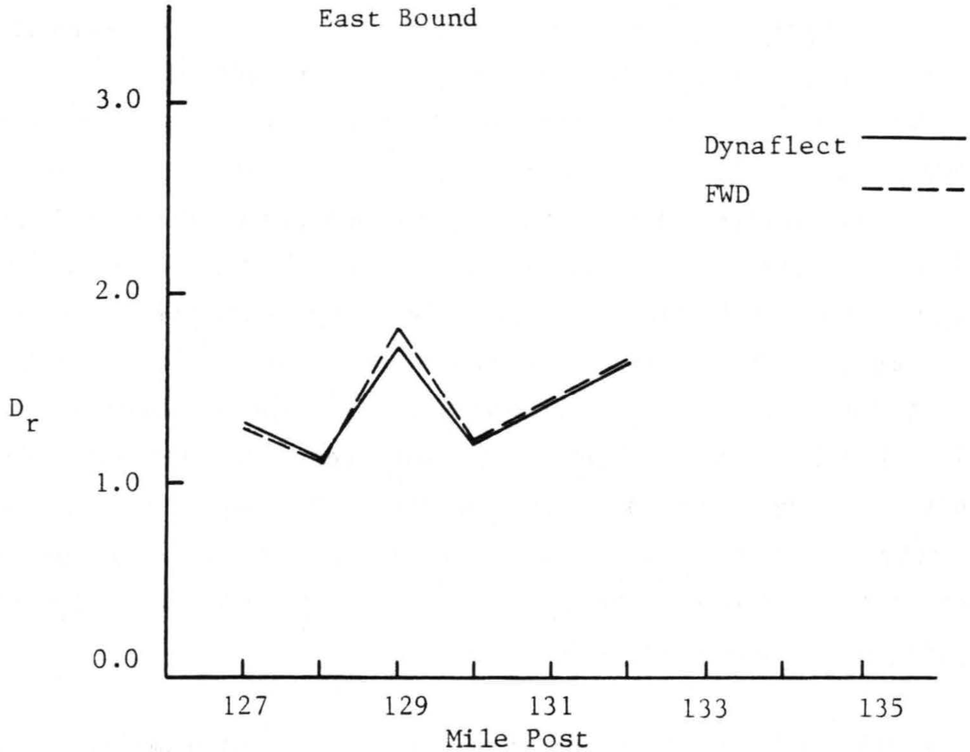
SITE NO.	NDT DEVICE	YOUNG'S MODULI (PSI)			
		P.C. CONCRETE	BASE	SUBBASE	SUBGRADE
1E	Dynaflect	3,543,991	233,416	143,961	21,081
	FWD	3,941,096	199,218	78,907	27,472
1W	Dynaflect	4,000,000	284,043	49,871	20,027
	FWD	4,367,000	639,820	300,000	20,290
5N	Dynaflect	3,028,750	107,000	105,250	27,430
	FWD	3,102,250	56,825	84,625	32,478
5S	Dynaflect	3,209,000	93,600	87,500	25,247
	FWD	3,108,000	52,100	100,950	31,703

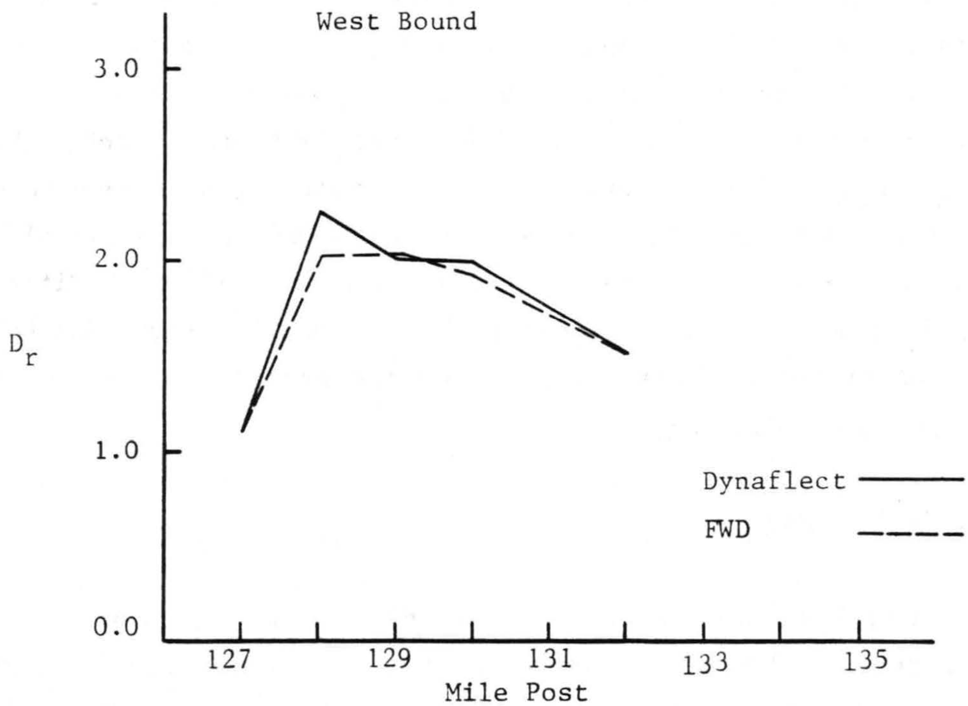
subgrade and surface concrete layer which are in close agreement. The moduli of intermediate layers for the Dynaflect and FWD, are not very consistent. However, structural response of rigid pavement is likely to be insensitive to variations in Young's moduli of intermediate layers (Ref 6). Similar findings are observed from the results of Site no. 5 except for the average subgrade modulus which is 15% to 20% higher for the FWD than the value computed from the analysis of the Dynaflect deflection basin. Surface concrete moduli are relatively higher for Site no.1 as compared to those of Site no. 5. The stiffness of the hot sand asphalt at Site no. 5 is low for a stabilized layer, and when comparing with Site no. 1. Possibly the base layer is a problem area. Remaining life estimates for both sites indicate that these pavements are in need of minor rehabilitation. But there is no indication of any significant structural deterioration of the pavement sublayers.

An indication of load transfer efficiency at transverse joints can be obtained by examining the ratio of Sensor 1 deflections at joint to midspan as illustrated in Figure 5.8. If this ratio approaches in the range 2 to 3, the load transfer at the joint is estimated to be poor with respect to midslab support. In Figure 5.8, FWD data (broken lines) measured at around 9000 lb. load are considered. It is interesting to note that the plots for both devices are approximately similar although the Dynaflect is a light load device and there is a significant difference in the loading modes of these devices. As shown in Figure 5.8 (a) the joints in the eastbound lanes seem to perform better than those in the westbound lanes for Site no. 1.

Flexible Pavements

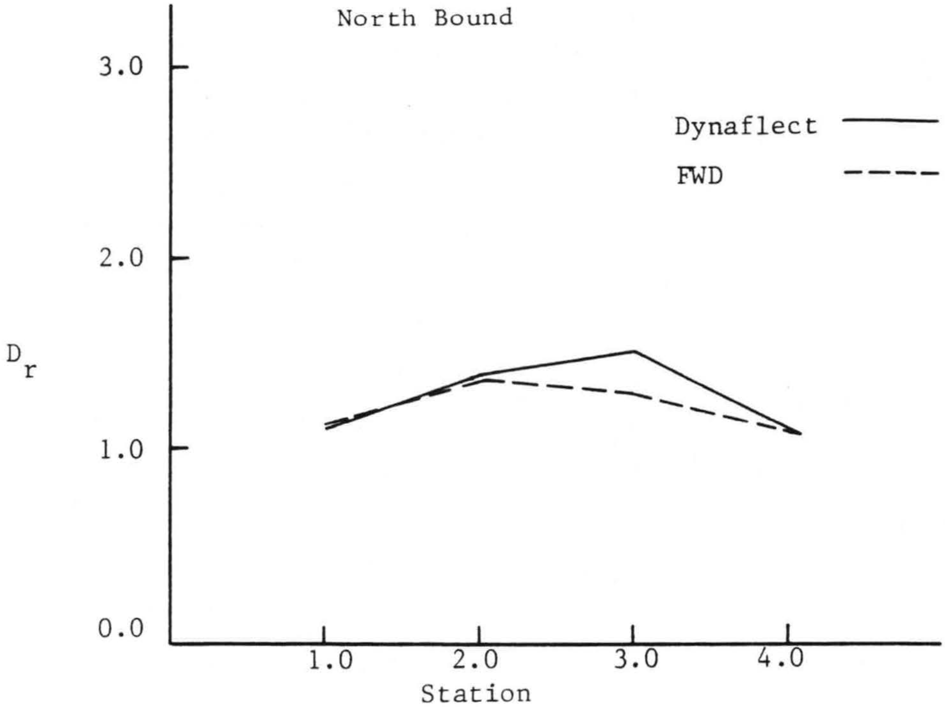
Results of Site no. 2 show that in general the moduli of asphaltic concrete layers at the test temperature are relatively low for surface layers. Remaining life estimates for the Dynaflect and the FWD indicate that generally fatigue cracking is not severe at this site. Pavement at Site no. 2 is in need of rehabilitation but the problem seems to lie in

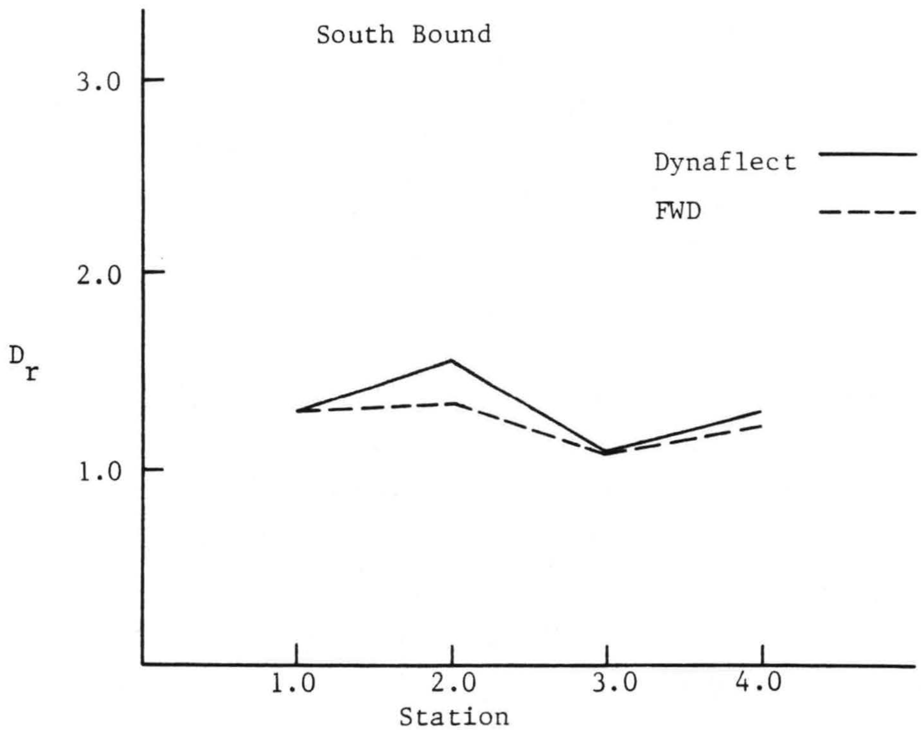




(a) Site 1

Figure 5.8. Variation of the ratio of Sensor 1 deflection at joint to that at mid span (D_r) for outside lanes.





(b) Site 5

Figure 5.8. (Continued)

the quality of asphaltic concrete material. Very large Sensor 1 deflections have been measured at many locations on Site no. 2 which repeatedly presented difficulties in obtaining a close match of the theoretical and the measured basin. Both the Dynaflect and the FWD basins showed this type of behavior. Table 5.10 presents summary statistics for moduli estimates of the flexible pavement sites.

Comparable results are obtained for surface AC and base moduli at Site no. 3, from the analyses of the FWD and the Dynaflect basins. Substantial difference is noted between the average subgrade moduli determined from the analyses of basins measured by the two devices. Lower subgrade moduli (around 30%) are obtained from the Dynaflect basins, as compared to the subgrade moduli from FWD basins. Existing structural capacity (mean value) of westbound lanes is almost twice that of the eastbound lanes.

Site no. 4 needs special attention. The average remaining life estimates are below 40% for both indicating possible fatigue failures. For the purpose of structural analyses the selected fill layer was divided into two layers. The average moduli of the bottom layer is lower than the average subgrade moduli for both devices. This indicates probable deterioration of the selected fill layer and its influence on surface AC layer as exhibited by relatively lower moduli of AC layer. Another interesting observation is that the average subgrade modulus from the Dynaflect test is remarkably lower than the average subgrade modulus from the FWD test.

For Site no. 6, the Dynaflect data were collected only in the northbound lane. However, the FWD data are available in both directions. The results indicate that pavement at this site is in good structural condition with respect to fatigue failure and Young's moduli of surface, base, and subgrade layers are consistently typical of good quality pavements. In addition to; the average subgrade modulus for the FWD at this site also is relatively higher than the average subgrade modulus

Table 5.10. Summary of average in situ moduli for flexible pavement sites

Site No.	NDT Device	Young's Moduli (psi) A.C.* Surface	Base	Subbase	Subgrade
2N	Dynaflect	384,031	205,415	--	26,230
	FWD	139,208	214,938	--	28,666
2S	Dynaflect	133,569	156,862	--	19,044
	FWD	96,977	146,538	--	17,911
3E	Dynaflect	268,306	95,854	--	18,666
	FWD	365,417	86,739	--	26,801
3W	Dynaflect	326,912	126,475	--	20,363
	FWD	433,267	211,233	--	29,347
**4S	Dynaflect	133,173	26,251	21,576	22,972
	FWD	164,667	23,312	13,515	31,360
6N	Dynaflect	696,738	234,963	--	19,213
	FWD	471,390	311,060	--	29,615
6S	FWD	558,333	278,922	--	31,572
7N	Dynaflect	834,186	162,771	29,514	16,426
	FWD	556,157	461,871	24,400	13,049
7S	Dynaflect	937,157	173,257	27,314	13,981
	FWD	447,729	506,771	25,171	12,636

* At test temperature

** Base and subbase are actually top half and bottom of a 24 in. selected fill layer.

obtained from the Dynaflect basins. This pavement had been overlaid. The modulus of base (asphaltic concrete) layer is remarkably lower than the total surface layers.

Site no. 7 is characterized by a variable finite thickness of subgrade. Evaluation of both the FWD and the Dynaflect basins has resulted in relatively large and consistent moduli for surface and base layers and no evidence of fatigue cracking. The analyses of the FWD in fact showed compressive horizontal strain at the bottom of the surface AC layer for several basins as indicated by zero values in Tables F-22 (b) and F-23 (b). The pavement on this site is in satisfactory structural condition. It should be recognized that an assumption of semi-infinite subgrade would have resulted in a significant overestimation of the subgrade modulus and subsequent errors in the computation of structural response.

Remaining life computations are based on fatigue analysis. In the real world, pavements can also fail due to excessive rutting. Therefore, it is important to look into the condition survey record and field investigations before making final conclusions about the structural condition of flexible pavements.

Composite Pavement

The results of Site no. 8, as expected, show compressive horizontal strains at the bottom of AC overlay as indicated by zero values in Tables F-24 and F-25. This means that there is no fatigue failure in AC layer. The moduli of subgrade for the Dynaflect are consistently lower than those for the FWD. Surface AC moduli are relatively lower when compared with those for typical mixes. Analyses of Dynaflect deflection basins have resulted in lower moduli for the sand cushion (subbase) layer than the moduli of subgrade. Moduli of the concrete layer are also relatively lower (50%) than the typical modulus for a good quality concrete (4 million psi).

SUMMARY

The Dynaflect and the FWD deflection basins collected in this study are summarized in this chapter. The selected basins measured in midslab position (for rigid pavements) and in wheel path (for flexible pavements) have been evaluated individually for in situ material characterization, and subsequently for structural response analyses. Results and summary statistics for each site have been presented in appropriate tables.

CHAPTER 6
DISCUSSIONS OF RESULTS

In order to hypothesize failure mechanisms present in each of the six sites of poor performance and the apparent success of the two sections which are performing well, various information regarding these sites were collected. This information includes interviews and evaluations by the research project selection committee and division engineers, diagnostic evaluations of the expert team, laboratory testing and analysis of both rigid and flexible pavement specimens, and nondestructive deflection testing and analysis. This chapter is devoted to the discussion of the results obtained from the above mentioned sources for each of the eight project sites.

SITE SPECIFIC DISCUSSION OF RESULTS

Site no. 1

This site was selected to represent the rigid pavement site which has performed as intended. This project is well designed and constructed. This 14 year old site carries heavy truck traffic of about 700 to 800 trucks in an 8-hour period weighing 70 to 80 thousand pounds. 130,000 pound overloads are common with incidents up to 151,000 pounds. Slabs are generally in good condition with relatively small spalling and cracking. Faulting is evident throughout the project and is more significant in the locations of increased fills. The edge joints and joint seals are in fair condition.

For this site, the analyses of Dynaflect and FWD data generated Young's moduli of elasticity of subgrade and surface layers which are in close agreement. Remaining life estimates indicate that the sublayers are in good structural condition but the pavement requires minor rehabilitation. No laboratory experiment was performed for this site.

Site no. 2

This is one of the flexible pavement sites which has performed poorly. There is a large amount of patching. Severe rutting and longitudinal cracks are present throughout the entire project although the magnitude varies. There is also some ravelling, surface wear, and transverse cracking.

Analysis of the Dynaflect and FWD data show that the moduli of elasticity of asphaltic concrete layers at the test temperature are relatively low for a surface layer. The remaining life estimate for this site indicates that the fatigue cracking is not severe. Although the pavement needs rehabilitation, the problem seems to lie in the quality of asphaltic concrete material.

Laboratory testing showed relatively lower indirect tensile strength for conditioned samples compared to dry samples, indicating a high degree of water susceptibility of the AC mix. Visual observation also showed 40 percent stripping of dry samples but about 100 percent for conditioned samples, indicating high moisture susceptibility. The Texas boiling test is not very supportive of the above findings.

Site no. 3

This is also one of the flexible pavement sites which has not performed satisfactorily. A small amount of surface wear and severe rutting is apparent. The subgrade is sandy and silty soil, with no evidence of differential movement. Rutting may be occurring in the upper layers because the soil is sandy and the site is located in a dry climate.

Analyses of FWD and Dynaflect basins produced comparable results for the asphalt concrete surface pavement. A 30 percent lower average subgrade modulus was obtained from the Dynaflect basin compared to the FWD basin. The mean value of the existing structural capacity of the westbound lanes is almost twice that of the eastbound lanes.

Lottman tests of specimens from this site showed no visual stripping in dry samples and only 20 to 30 percent stripping in the conditioned samples, indicating some moisture susceptibility of the AC mixture. However, very low tensile strength in conditioned samples from the bituminous base indicates severe stripping in the base.

Site no. 4

This is one of four flexible pavement sites which showed poor performance. This section has a large amount of ravelling and rutting. Longitudinal, transverse, fatigue, and block cracking are also present. This roadway has a high percentage of heavy trucks.

The selected fill layer was divided into two layers for structural analyses of the Dynaflect and FWD basins. The average modulus of the bottom layer is lower than the average subgrade modulus indicating deterioration of the selected fill layers and its influence on the asphalt concrete surface layer as exhibited by lower moduli of the asphalt concrete layer. Dynaflect tests resulted in lower average subgrade modulus compared to FWD test. The average remaining life below 40% is indicating possible fatigue failure.

Laboratory experiments indicated very low moisture susceptibility of the asphaltic concrete mixture from Lottman tests. Texas boiling test also supports the above finding. Percent air voids are generally satisfactory.

Site no. 5

The site was selected to represent the rigid pavements which have not performed satisfactorily. Ride quality is very poor due to the faulted slab joints. The edge joint is poor and the joint seal condition is fair.

For this site also, analyses of both Dynaflect and FWD data produced Young's moduli of elasticity for subgrade and surface layers which are in close agreement. The remaining life estimate for this site indicates that the pavement requires major rehabilitation.

Split tensile strength tests show uniform and satisfactory concrete strength. Tensile strength tests on hot sand asphalt samples show no visual stripping for the dry sample and only 20 to 30 percent in the conditioned samples which indicates low moisture susceptibility. Texas boiling test also is in support of the above finding.

Site no. 6

This is also one of the four flexible pavement sites which did not perform satisfactorily. This has been overlaid with 2" ACP and an open graded friction course. This roadway carries a large amount of traffic and appears to have a high percentage of heavy trucks. Rutting and incipient bleeding was observed throughout the entire project.

Dynaflect and FWD tests indicates that the pavement is in good structural condition in terms of fatigue failure. Also the Young's moduli of elasticity of surface and subgrade are typical of good quality pavements. However, the modulus of base asphaltic concrete layer is substantially lower than that of the surface layer.

Lottman tests on the surface AC mix show severe stripping and thus high moisture susceptibility. Results of the Texas boiling tests are also in agreement with this finding.

Site no. 7

This site was selected to represent the flexible pavement sites which have performed satisfactorily. This roadway has a very smooth ride and very little truck traffic. A very small amount of rutting was observed.

Evaluation of both FWD and Dynaflect basins produced large values of modulus of elasticity for both surface and base layers and did not show any evidence of fatigue cracking. The pavement is in satisfactory structural condition.

Lottman test results show very little moisture susceptibility of the AC mix. The percent air voids values indicate low variability of the mix. The modulus of resilience values are also higher compared to site 2 and 4.

Site no. 8

This site was selected to represent composite pavement sites which have failed to perform satisfactorily. On the asphalt concrete pavement overlay sections, large number cracks or joints have reflected through. Excessive rutting and some longitudinal cracks were also observed. Some of the causes of distress could be water holding of the open graded friction course which stripped the top type C ACP.

Dynaflect and FWD data show no fatigue failure in the asphalt concrete layer. Surface AC moduli are relatively lower compared to typical mixes. The sand cushion subbase layer has lower moduli compared to that of the subgrade. Also moduli of concrete layer is about 50 percent of that of a good quality concrete.

Split tensile strength tests for the Portland cement concrete layers show uniform and satisfactory concrete mixture. Both Lottman and Texas boiling test of specimens from the asphalt layers show substantial amount of stripping indicating the potential moisture susceptibility.

SUMMARY

Based on the field observations, laboratory investigations, and nondestructive pavement evaluations of rigid, flexible, and composite pavements it is concluded that stripping of the asphalt concrete mixture is the main reason for premature distress. In addition to stripping there are some site specific limitations which are acting as compounding factors in enhancing the failures.

The major distress observed in the failed rigid pavement site is faulting. Both nondestructive pavement evaluation and laboratory investigations indicated that the PCC slabs are in good structural condition and are constructed with proper mix design. The reason for the faulting of the PCC surface pavements are mainly due to stripping of the AC mix in one of the underlying base layers. In addition to stripping, larger differential deflection is also partially responsible for faulting.

Rutting, shoving, longitudinal and transverse cracking were the most commonly observed distress in all the five failed flexible pavement sites. Once again, stripping of the asphalt concrete mixture in one of surface or base layers can be blamed for the above mentioned distress in the flexible pavement sites. An asphalt concrete layer loses its stability substantially and thus causes shear failure in that layer. Rutting and shoving of the surface layer is caused due to the shear failure of an underlying layer. For some of failed flexible pavement sites, overloading of highways beyond the designed capacity could be responsible to some extent for the observed distress.

For the composite pavement site, rutting and reflection cracking are the main failures observed. Laboratory investigations concluded that, the PCC layer maintains adequate structural capability but stripping is occurring in the asphalt concrete layers. Thus shear failure of one of the underlying layers due to the stripping is the cause of rutting and reflection cracking in the surface layer.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

It is concluded that the most likely cause of premature distress in both the rigid and flexible pavements is due to the stripping and/or loss of strength due to poor moisture susceptibility characteristics. The effect of problems with the asphaltic mixture is compounded by the heavy loads reported to be travelling on several of the projects included in the 8 sections evaluated. It is significant to note that the one project with relatively low volume of truck traffic, site no. 7, is maintaining a high level of performance.

The overall structural design would appear to be satisfactory if the materials could retain their full strength during periods when moisture is present in the asphalt mixture.

RECOMMENDATIONS

Phase I of this study resulted in the following recommendations for possible rehabilitation of project sites and evaluation of the pavement design procedure used by the State of Oklahoma, Department of Highways:

Flexible Pavements

1) Re-evaluate mix design requirements
   a) Consider the use of tensile strength requirements using a split tensile test procedure
   b) Consider the use of a creep test to evaluate potential rutting problems.

c) Establish requirements for water sensitivity, e.g., Lottman test for retained strength. Discussions with ODOT personnel indicate this is under study or is being implemented.

2) Consider the use of hydrated lime to correct stripping and water sensitivity problems. Approximately 1.5 percent, by weight of mix, has proven to be universally beneficial with regard to improving water sensitive strength tests. Chemical additives can also be used; however, their effectiveness needs to be carefully evaluated by laboratory tests with the aggregate and asphalts planned for use on a specific project.

3) Require a harder asphalt on heavily trafficked highways. ODOT is currently using an AC-20 (AASHTO Table 2) asphalt cement in asphalt concrete. Since heavy duty, full-depth, asphalt pavements have low deflections, fatigue or alligator cracking is less of a concern. Research studies show that harder asphalts can be used in thick asphalt concrete pavements. The use of an AC-40 should be considered.

4) Consider increasing the percent crushed aggregate asphalt concrete on heavy duty highways, i.e., ADT greater that 5000. The present requirement of 70 percent crushed on coarse aggregate could be modified to require 85 percent crushed on the combined aggregate, coarse and fine.

Rigid and Composite Pavements

Failures of both rigid and composite pavement sites can be corrected primarily by improving the mixture of the underlying asphalt concrete layers by following the previous recommendations made for flexible pavements. In addition to that, for rigid pavements some type of load transfer between the slabs is needed, edge joints should be sealed and continually monitored to ensure that the seal is maintained. For composite pavement sites consideration should be given to designing a stress relieving layer that will reduce reflection cracking from the joints of concrete pavement, which was overlaid. A possible stress relieving layer could be fabric or an Arkansas mix.

Site Specific Recommendations

The following set of recommendations have been made with regard to possible action for each of the projects studied. These recommendations should be considered preliminary; final determinations will depend on more detailed engineering investigations. Nevertheless, the type of action recommended are considered generally appropriate for each site.

Site no. 1

This site has performed satisfactorily. Due to the heavy overload of traffic, the pavement requires minor rehabilitation indicated by remaining life analysis.

Site no. 2

The precise thickness of the material to be removed should be based on further investigation as to the depth of the unstable layer, i.e., stripping or poor moisture susceptibility properties. The removed top layer of asphalt concrete should be replaced with the same thickness of virgin mix using slurry lime to modify properties of aggregate. Use an AC 40 as binder. Modify aggregate gradation to limit the amount passing the No. 4 sieve to 50 percent.

Site no. 3

An overlay may be satisfactory if stripping is not present in one of the layers. If stripping is present, then consideration should be given to removing the problem layer.

Site no. 4

Remove upper 2-3/4 inches of asphalt concrete. Replace with the same thickness of virgin mix using slurry lime to modify properties of aggregate. Use an AC 40 as binder. Modify aggregate gradation to limit the amount passing the No. 4 sieve to 50 percent. Use of OGM is considered optional.

Site no. 5

Correction of the excessive faulting on this project will be difficult since milling, for example, will only be a temporary measure. It appears that some type of load transfer devices concentrated in wheel path are needed. Also edge joint should be sealed, and a policy to permit a continued monitoring to insure the seal is maintained. Subsealing at the joints should be considered in order to fill the void and establish continuity between slab and surface.

Site no. 6

Unless rutting is considered unsafe, it should be possible to defer rehabilitation for several years.

Site no. 7

No action required.

Site no. 8

Remove asphalt concrete; determine amount of voids under PCC, subseal to establish compatibility with subbase. Design a stress absorbing layer over PCC and surface with a minimum of 4 inches of asphalt concrete meeting requirements for Sites 2 and 4.

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APPENDIX A

DETAILS OF INTERVIEWS AND EVALUATIONS

QUESTIONS PRESENTED TO RESEARCH PROJECT SELECTION COMMITTEE

1. What are the reasons for the selection of the eight pavement sites?

2. What major distress and maintenance problems are experienced in the state?

3. What pertinent data, such as condition surveys, pavement construction and performance histories, traffic soil strengths, and environmental histories are currently available in ODOT files?

4. What pavement design procedures and management practices are currently being followed?

5. What are the problems and/or inadequacies with current pavement design procedures and management practices?

QUESTIONS ON ROAD SITES PRESENTED TO DIVISION ENGINEERS

1. What distress was observed?

2. What are the causes of distress?

3. What are the recommended method of repairing the distressed pavement?

4. Provide any records, diaries of construction, maintenance, weather information, or other data which might be helpful in

determining the cause of premature pavement failure for the section or sections in your division.

5. What are the major causes of pavement failures at other sites in your division?

6. What major maintenance practices are used in your area?

RESULTS OF DIVISION ENGINEERS INTERVIEWS

Site Number 1 - I-40 Canadian County - From 2 1/2 miles west of S.H. 92 in Yukon west approximately 7 3/4 miles, just past the U.S. 81 Interchange, rigid pavement, satisfactory performance.

The project was well designed and constructed by a good contractor. It has 15 foot sawed joints with no dowels and was placed with a slip form paver. The subgrade west of US-81 for about 1/2 mile and the next 1 1/2 miles was rather bad. This may have contributed to minor faulting of the joints. However, for a freeway approximately 14 years old and carrying the heavy truck traffic, it is in very good condition.

While visiting the weigh station at this site, it was learned that 700 to 800 trucks weighing 70 to 80,000 pounds came through in an 8-hour shift. 130,000 pound overloads are common after closing the weigh stations or avoiding same on a parallel highway. One recent load picked up was 151,000 pounds (heavy earth-moving equipment).

Site Number 2 - US-69 from US-270 in McAlister North approximately 5 miles to SH-113, flexible pavement, unsatisfactory performance.

1) The distress observed by personnel is rutting and crumbling.

2) They think the cause is not enough asphalt in the mix and asphalt not as good as it used to be. The asphalt came from Muskogee and

and the aggregate came from Youngman at Onapa, Oklahoma. They have used it before without problems but it is a highly absorptive sand stone.

3) Suggestions for repairing this distressed pavement would be to recycle it if the material is good enough. The project consists of an 8 inch type A, 2 inch type C, and 3/4 inch of OGFC. They like the friction course very much for safety. Regarding any records, the Oklahoma State office has all the information. They think stripping is occurring in the Type A mix. Any repair or maintenance performed on this section was primarily to remove with a back hoe and replace it. Also, a fog seal was placed last year.

<u>Site Number 3</u> - Interstate Highway 40 from 1/4 mile east of S.H. 30 Interchange in Erick west approximately 7 1/2 miles to the Texas State Line, flexible pavement, unsatisfactory performance.

Distress observed:

1) The distress observed was rutting in the driving lane and some cracking but the cracking was considered no problem. The cause of this distress is that it is located in a sandy area, and there was heavy loads and overloads and they think that the expansion and contraction of the pavement may have caused some of the distress. Heavy rutting occurred in the last two summers. They think that the pavement design procedure is okay. If the present loads are maintained; if possible, they need to increase the stability of the fine aggregate asphalt mixture. In this project the grading was established and set a while in this sandy area. The type C asphaltic concrete is considered to be good. They think the hot sand is the culprit and possibly it needs screenings added to it. The minimum stability of 17 required then should be raised to

probably to 21 or more. Insofar as number 3, how to repair it, they suggest roto-milling to the base of the rut and an overlay with a type B or C and then go back with a popcorn surface. Also they think that the popcorn is good for stability and gets rid of the water. They indicated that plans are nearly ready for correcting the rutting and they will send us information on this. This is a temporary solution where they are going to use rolumac which is a rapid setting emulsion and placed by the slurry method.

2) Historical records will be furnished by the state lab.

3) Insofar as the pavement design procedures, they think that possibly the pavement design procedure allows underdesigned pavements. This project was built before loads were increased and one of the most important things is to have shoulders for lateral support.

4) Insofar as maintenance, seal coats are not used. Pouring the joints with rubber and some fog sealing is done in the division. This project was completed in 1975 and is approximately 9 years old. This particular project won the 1975 national honors for full depth asphalt paving from NAPA. It had already won first prize in the Oklahoma Asphalt Pavement Association competition.

<u>Site Number 4</u> - US-69 south of Caney, north approximately 7 miles to near Tushka, flexible pavement, unsatisfactory performance.

1) Distress observed by the group included rutting, some distortion, and from past observations, stripping. The rutting was considered to be due to the asphalt yielding.

2) The cause of the stripping was water entering and penetrating from the top to the bottom causing stripping of the asphalt base.

Coring showed stripping in the various layers. Some thought the moisture in the mix was not removed by the drum drier plant.

3) Recommended methods of repairing was to cut out and replace with a dense mix or recycle and overlay.

4) Records were provided for the project.

5) Was not answered.

6) Major maintenance in this area is blade leveling, patching, or digging out and relaying the asphalt in various sections.

7) General - Maybe the failure was caused by water entering the mix before it was completed and opened to traffic. Some suggestions were to put an additive in to prevent stripping and more control of moisture in the asphalt mix at the plant. There was a statement that they would never put open graded friction course on a finished job again. The subgrade here had a zero plasticity index and suggested that the density should be at least 95 percent at standard density (AASHTO T180, Method D).

<u>Site Number 5</u> - US 69 from north of Checotah Interchange north approximately 5 miles to the Oktaha Interchange, rigid pavement, unsatisfactory performance.

1) Distress observed is slab faulting and pumping at longitudinal and transverse joints.

2) Cause of the distress is leakage and possibly a poor sealing compound. Joint sealing material consisted of two component polymer and they were thinking that they needed a center line joint.

3) Recommended method of repairing this was to build the pavement with sawed joints only and a directive was given last week to do this, but possibly this job should be recycled - either by breaking it up and overlaying it or crushing the material and making new concrete pavement. Possibly they could grind the joints to smooth them down.

4) Records will be provided by the state office. The perception of major causes of pavement failures in the area was due to underground water, clays, and shales. Insofar as designing this project again, they would get rid of the low stability sand asphalt and even if you did not have stripping, the hot sand asphalt is not strong enough. They suggested the possibility to go back to sixty foot joints and do not recommend placing dowels because they cannot be placed properly. Possibly they should use wire mesh. Specifications require maximum 1/2 inch aggregate and the subgrade here is shale, clay, and silty loam and all materials were lime stabilized. The asphalt came from the Onapa refinery. Standards from the 1960's were for 15 foot joints.

<u>Site</u> <u>Number</u> <u>6</u> - I-35 from 1/2 mile north of the S.H. 59 Interchange, south approximately 5 miles, flexible pavement, unsatisfactory performance.

1) No interview was conducted on this project because originally it was believed to be a good section. However, it is noted that some rutting is occurring throughout this project. It was later learned that this site had been overlaid and an open graded friction course was also placed.

Site Number 7 - US 75 from north of Copan, 6 miles south of the Kansas State Line, north approximately 5 miles, low traffic, flexible pavement, satisfactory performance.

1) This is also a very good section and the reasons for it was good materials, good contractor and proper design. Additionally, this was a turnkey job or a button-up job and it had a sandy low PI subgrade in this area which provides a good building site. It had a good mix design, had good maintenance, the oil came from Tulsa and is pretty consistent asphalt. The aggregate was from Leco Materials, in Dewey, Oklahoma type A. They have had good luck with sand from Sand Springs which is washed sand. The A.C.P. consists of special chat added from Arrowhead and they had an insoluble residue requirement. They had good resident and construction engineers. Insofar as design, they are not satisfied with the design procedure because this is done at the state office by personnel who have not been on the job. Also the local division has little or no input. Local environment is not given much consideration. They give too much credit to the top lift. They need more subgrade treatment. A minimum of 1 foot is recommended.

Site Number 8 - IH 35 from a point 6 miles north of the US 64 Interchange in Perry and north approximately 11 miles, flexible pavement, unsatisfactory performance.

1) Distresses showed up the first of the spring. Shoving, rutting, in the outside lane was worse. Reflective cracking from the joints of the concrete pavement, which was overlaid, appeared the first of the fall and 100% of it is now there. The shoulder crack is coming through and they have had some potholes. This site was opened to traffic in the spring of 1980. The asphalt AC3 came from Trummell and Allied and the aggregate was Quapac from Drumright, Oklahoma. They have had no problems with it before. Most of the concrete came from Caw Industries. They repaired all

the old joints by removing about 4 feet of the concrete adjacent to the cracks and repouring it before overlaying.

2) Some of the causes of distress, they think the open graded friction course holds the water and stripped the top type C. They thought they had a high penetration oil and percent of the asphalt was too high. The popcorn overlay was laid at the same time as the type C.

3) Suggestions for correction of this facility would be to either remove the asphalt and recycle and add new material thicker, plus possibly the use of Petromat although they really do not think Petromat will stop the cracking. Another possibility would be to recycle the concrete or break it up and use as a base and then overlay it. They think possibly the highway may be worse in the northbound direction. Also they mentioned that one lane is thicker than the other near the south end. They think the asphalt is no good. The asphaltic concrete is no good and they do use anti-strip in the open graded friction course. They observed "d" cracking in the outside when they removed some of the concrete at the joints and they think this was due to the salt contamination and also due to the fact the joints were never resealed. They say that the usual maintenance practice here does not include resealing the joints and they do not reseal the shoulder joints. About all they do is fill the potholes with asphalt. This project nearly won second place in the NAPA Highway Contest. Some records were furnished as to the anti-strip agent which is Pavebond from Thiokol, Cincinnati, Ohio.

APPENDIX B

Table B-1. Results of pavement condition survey for Site No. 1.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Base Type	Base Thick(in)
I-40-4(50)127 Canadian County West of U.S. 81 East approx 7.6 mi.	1967	PCC	9	Bituminous base - fine aggregate type	4

Joint Seal Condition	Shoulder Seal	Cracking	Spalling	Faulting	Pumping	Surface Wear
			Westbound			
Fair to Poor	Poor	5-shattered slabs(2)	N.O.	0.25	N.O.	N.O.

Shoulder Drop Off (inches)	PSR	Overall Rating	Traffic
			Estimated in 1967 (See memo from Perry 2-24-67 ADT = 24038 % trucks = 6

(2) Shattered slab - broken into four or morre parts; a local condition problbly due to subsidence in area.

Table B-2. Results of Pavement Condition Survey for Site No. 2.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Binder Type	Binder Thick(in)	Base Type	Base Thick(in)
Pittsburg County U.S. 69 - from north of McAlester north approx. 3.1 miles to S.H. 113	1982	OGM	0.75	C	2.0	A	8

Pavement Condition: Rutting (inches)	Cracking: Alligator	Cracking: Transverse	Cracking: Longitudinal	Raveling	Bleeding

Southbound

This project is exhibiting distress of all types with the possible exception of raveling. The maintenance forces of ODOT have completed extensive repairs to the project (50% of area) and some of these repairs are beginning to rut and bleed. Alligator cracking and pumping were noted in some of the areas which have not been repaired.

Northbound

This project (Asphalt concrete overlay) is exhibiting some rutting; however, the major form of distress is the transverse (reflection) cracks from underlying PCC pavement. In one 500' area, the average transverse crack spacing was approximately 50 feet and varied from 15 feet to 100 feet.

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
		Northbound Very poor	Estimated in 1981 (See memo from Cuaderes 3-31-81 ADT = 11150 % trucks = 21	
		Southbound Fair		

Table B-3. Results of Pavement Condition Survey for Site No. 3.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Binder Type	Binder Thick(in)	Base Type	Base Thick(in)
I-40 Bechham County From Oklahoma-Texas State Line East 7.82 miles	1973	"C"	1.5	"A"	3.0	Bituminous Fine Aggregate	8-10

Rutting (inches)	Pavement Condition: Cracking Alligator	Cracking Transverse	Cracking Longitudinal	Raveling	Bleeding
Westbound					
0.5-1 O.L.(1) <0.5 I.L.(1)	N.O.(2)	Insignificant(3)	< 1%(4)	N.O.	Incipient in wheel path - OK only
Eastbound					
0.5-0.75 O.L. 0.25-0.5 I.L.	< 1%(4)	Intermittent(5)	< 1%	N.O.	Incipient in wheel

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
Westbound OK Raveling Low severity	3.5-4.0	Good	From plans ADT = 9695 % trucks = 15	
Eastbound OK	3.5-4.0	Good		

(1) O.L. = Outside Lane; I.L. = Inside Lane
(2) N.O. = Not Observed
(3) Two transverse cracks of low severity in four stops made in the direction
(4) Low severity, less than 1 percent of section length
(5) Low severity, no regular pattern

Table B-4. Results of pavement condition survey for Site No. 4.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Binder Type	Binder Thick(in)	Base Type	Base Thick(in)
Atoka County U.S. 69 - from Caney north approx. 5.9 mi. to Tushka	1981	OGM	0.75	--	--	B	8-10
		OGM(1)	0.75	B	4 plus petromat	--	--

Rutting (inches)	Pavement Condition: Cracking Alligator	Cracking Transverse	Cracking Longitudinal	Raveling	Bleeding
Southbound					
0.5-1.5 O.L. 0.25-0.75 I.L.	Significant amounts- a range of 5% to 100% of length of section based on 8 - 500 ft sections evaluated	N.O	Significant amounts - ranges from 5% to 50% of length of section based on 8 - 500 ft sections evaluated	Ranges from low to medium severity using PCI scale	Not Significant- localized sections in short lengths
Northbound					
< 0.5	N.O.	Reflection cracks from underlying PCC - would appear that 100% are reflecting through	See Transverse cracking	N.O.	N.O.

Table B-4. Results of pavement condition survey for Site No. 4 (contd).

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
Southbound				
OK	3.0-3.5	Poor	Estimated in 1976 (See memo from Caudares 3-2-76) ADT = 10700 % trucks=21	
Northbound				Skin patches in southerly portion of project

(1) No evidence of OGM in Northbound lanes

Table B-5. Results of pavement condition survey for Site No. 5.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Base Type	Base Thick(in)
U.S. 69 McIntosh-Muskogee on U.S. 69 north of I-40	1978	PCC	9	Bituminous base fine aggregate type	4[a]

Joint Seal Condition	Shoulder Seal	Pavement Condition: Cracking	Spalling	Faulting	Pumping	Surface Wear
Northbound						
Fair to Good[1]	Poor	N.O.	N.O.	0.25	N.O.	N.O.
Southbound						
Fair to Poor	Poor	N.O.	N.O.	0.25-0.75	N.O.	N.O.

Shoulder Drop-off (inches)	PSR	Overall Rating	Traffic
Northbound < 0.5	2.5-3.0	Good	Estimated in 1973 (See memo from Gauderos 1-29-73)
Southbound			ADT = 13200 % trucks = 21

(a) 6 inches of lime modified subgrade in selected sections
(1) Good-joint is sealed; Fair-joint is partially sealed; Poor-joint is not sealed

Table B-6. Results of pavement condition survey for Site No. 6.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Binder Type	Binder Thick(in)	Base Type	Base Thick(in)
I-35 McClain County From Garvin-McClain County Line north to SH59	1969 overlay in 1979	OGM	0.75	A	3	Plant Mix bituminous base course	13-14

Rutting (inches)	Pavement Condition: Cracking Alligator	Cracking Transverse	Cracking Longitudinal	Raveling	Bleeding
Southbound					
0.5-0.75 O.L. 0.25-0.5 I.L.	N.O.	N.O.	Insignificant along outer edge of OGM	N.O.	Insignificant
Northbound					
0.25-0.5 O.L. 0.25 I.L.	N.O.	N.O.	N.O.	N.O.	Insignificant

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
OK	3.5-4.0	Good	Estimated in 1976 (See memo from Perry 10-30-67) ADT = 17250 % trucks=11	

Table B-7. Results of pavement condition survey for Site No. 7.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Type	Wearing Thick(in)	Binder Type	Binder Thick(in)	Base Type	Base Thick(in)
Washington County U.S. 75 from Kansas Oklahoma State Line South 1.9 miles	1978	C	1.5	A	3.0	Plant Mix-Coarse Aggregate	14

Pavement Condition: Rutting (inches)	Cracking: Alligator	Cracking: Transverse	Cracking: Longitudinal	Raveling	Bleeding
Northbound					
0.25	N.O.	N.O	N.O.	N.O.	N.O.
Southbound					
0.25	N.O.	N.O.	N.O.	N.O.	N.O.

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
OK	4.0-4.5	Excellent	From plans ADT = 4850 % trucks = 10	7 trucks in 35 minutes, both directions
OK	4.0-4.5	Excellent		

Table B-8. Results of pavement condition survey for Site No. 8.

Project Description	Date Opened to Traffic	Pavement Composition: Wearing Surface Type	Wearing Surface Thickness(in)	Base Type	Base Thickness(in)		Joint Seal Condition
Noble County I-35 from Cimmaron Turnpike to Junction of SH15 north of Perry	1980	OGM	0.75	C	2.0	PCC	9

Rutting (inches)	Cracking: Alligator	Cracking: Transverse	Cracking: Longitudinal	Raveling	Bleeding
Northbound					
0.25-1.5 O.L.	N.O.	Reflection cracks from PCC, both from joint repairs & mid-slab cracks	N.O.	Some wear in O.G.M. not considered significant	N.O.
Southbound					
0.25-1.5 O.L. 0.25 I.L.	N.O.	Reflection cracks from PCC, some cracks, unsealed up to 1 in. wide	N.O.	N.O.	N.O.

Shoulder Condition	PSR	Overall Rating	Traffic	Comment
Transverse cracks from soil-cement base				Maintenance has corrected localized corrugations and shoving by removing high points with planer
Transverse from soil cement base	2.5-3.5	Fair to Good		

APPENDIX C

Table C-1. Sample identification of laboratory experiments

Sample No.	Site No.	Description
1	8	Southbound
2	8	Southbound
3	3	Top, 5.0 mile
4	3	Bottom, 5.0 mile, ATB
5	4	Bottom, 3.0 mile
6	4	Top, 1.0 mile
7	7	Top, 2.0 mile
8	8	Northbound, Upstream, 8.2 mile
9	8	Northbound, Downstream, 8.2 mile
10	4	Bottom, 6.0 mile
11	3	Eastbound, Top, 0.8 mile
12	3	Top, 0.35 mile
13	3	Bottom, 0.35 mile
14	3	Eastbound, Top, 7.7 mile
15	3	Eastbound, Bottom, 0.8 mile, ATB
16	3	Eastbound, Bottom, 7.7 mile, ATB
17	2	Southbound, Top, 1.4 mile
18	2	Southbound, Bottom, 1.4 mile
19	2	Northbound, Bottom
20	5	Southbound, 3 mile, ATB
21	5	Northbound, 2 mile, ATB
22	5	3 mile, ATB
23	5	2 mile, ATB
24	2	Southbound, 3.6 mile, AC
25	3	0.35 mile, Sand Asphalt, ATB
26	2	Middle, Southbound, 1.4 mile
27	3	5 mile, ATB
28	3	Eastbound, 7.7 mile, ATB
29	2	Top, Northbound, 1.8 mile
30	2	Top, Northbound, 1.8 mile
31	4	Bottom, 1.0 mile
32	4	Bottom, 1.0 mile
33	4	Top, 3 mile
34	7	2 mile
35	7	2 mile
36	7	Southbound, 1st lift, surface
37	7	Southbound, 2nd lift, Base
38	7	Southbound, 3rd lift, Base
39	7	Southbound, 4th lift, Base
40	7	Southbound, 1st lift, Surface
41	7	Southbound, 2nd lift, Base
42	7	Southbound, 3rd lift, Base
43	7	Northbound 1st lift, Surface
44	7	Northbound 2nd lift, Base
45	7	Northbound 3rd lift, Base
46	7	Northbound 1st lift, Surface

Table C-1 Sample identification of laboratory experiments (contd.).

Sample No.	Site No.	Description
47	7	Northbound 2nd lift, Base
48	7	Northbound 3rd lift, Base
49	7	Northbound 4th lift, Base
50	6	Southbound 1st lift, Surface
51	6	Southbound 2nd lift, Surface
52	6	Southbound 3rd lift, Base
53	6	Southbound 1st lift, Surface
54	6	Southbound 2nd lift, Surface
55	6	Southbound 3rd lift, Surface
56	6	Southbound 4th lift, Base
57	6	Northbound 1st lift, Surface
58	6	Northbound 2nd lift, Surface
59	6	Northbound 3rd lift, Base
60	6	Northbound 1st lift, Surface
61	6	Northbound 2nd lift, Surface
62	6	Northbound 3rd lift, Surface
63	6	Northbound 4th lift, Base
64	4	Southbound 1st lift, Surface
65	4	Southbound 1st lift, Surface
66	4	Southbound 2nd lift, Surface
67	4	Southbound 3rd lift, Surface
68	2	Northbound 1st lift, Surface
69	2	Northbound 2nd lift, Base
70	2	Southbound 1st lift, Surface
71	2	Northbound 1st lift, Base

Table C-2. Designation of laboratory tests.

Physical and Mechanical Properties	Test Designation
Bulk Density	ASTM D1188-71
Asphalt Content (%)	ASTM D2172
Effective Specific Gravity	ASTM D2041
Air Void (%)	ASTM D3203
Grain Size Distribution Analysis	ASTM C 136-82
Resilient Modulus	ASTM D 4123-82
Split Tensile Strength of Cylindrical Concrete	ASTM C 496-71

Table C-3 Asphalt concrete mix characteristics

Site No.	Sample No.	Bulk Density (lb/cu.ft.)	Asphalt Content (%)	Resilient Modulus at 72° F. $M_R, 10^6$ (psi)	Effective Specific Gravity	Air Void (%)	Sample Location
2	17	135				11.8	Top*
	18	138				10.2	Bottom*
	19	137				10.5	Bottom
	24	141					
	26	144			2.45	6.4	Middle*
	29	139				9.5	
	30	137				10.8	
	68	140		1.18			Surface
	69						Base
	70	140	5.52		2.31		Surface
	71	142	5.33		2.44		Base
3	3	157					Top
	4	121				19.2	Bottom
	11	152			2.50	2.5	
	12	153				1.8	Top
	13	127					Bottom
	14	152			2.50	2.1	
	15	123				17.4	Bottom
	16	125				16.2	Bottom
	25	117			2.40	21.4	
	27	124			2.40	16.8	
	28	154				16.2	
4	5	121				19.2	Bottom
	6	150				2.1	Top
	10	140			2.37	5.6	Bottom
	31	141				4.9	Bottom
	32	140	5.47		2.37	5.1	Bottom
	33	139				5.8	Top
	64	142	5.81		2.39		Surface
	65	144		1.97	2.38		Surface
	66	140	4.94	1.94			Surface
	67	136		0.97			Surface
5	20	120				14.2	Base
	21	132			2.43	12.7	Base
	22	131			2.43	13.7	Base
	23	131				13.4	Base
6	50	148	4.89	1.20	2.45		Surface
	51	146		2.12			Surface
	52	143	4.06		2.43		Base

* Base layer (based on qualified judgement and record of field notes).

Table C.3 Asphalt concrete mix characteristics (contd.)

Site No.	Sample No.	Bulk Density (lb/cu.ft.)	Asphalt Content (%)	Resilient Modulus at 72° F. $M_R x10^{-6}$ (psi)	Effective Specific Gravity	Air Void (%)	Sample Location
6	53	149	6.11		2.46		Surface
	54	145		2.69			Surface
	55	149	4.25		2.44		Surface
	56	140					Base
	57	144					Surface
	58	149	4.66		2.43		Surface
	59	140					Base
	60	150		1.95			Surface
	61	142		1.05			Surface
	62	145					Surface
	63	141					Base
7	34	145				5.9	
	35	145			2.47	5.9	
	36	149	4.13		2.44	2.5	Surface
	37	150		1.72		3.7	Base
	38	149				4.5	Base
	39	148	4.84		2.47	5.2	Base
	40	148				2.8	Surface
	41	151		2.49	2.55	3.3	Base
	42	150	4.45			4.2	Base
	43	146				4.1	Surface
	44	148				5.0	Base
	45	149	4.36		2.49	4.7	Base
	46	144	4.91		2.45	5.2	Surface
	47	146		3.79		6.2	Base
	48	147				5.7	Base
	49	149				5.0	Base
8	1	151			2.52	4.0	
	2	152				3.2	
	8	148				6.0	
	9	146			2.53	7.3	

Table C-4. Split tensile strength of cylindrical PC concrete cores.

Site No	Sample No	Split Tensile Strength (psi) Top Layer	Bottom Layer
5	72	576	617
	73	663	625
	74	656	618
	75	612	589
8	76	537	587
	77	465	493
	78	572	427
	79	583	469

Table C-5. Results of Tension Test of Dry Samples

Site No.	Sample No.	Tensile Strength (psi)	Description of Sample After Tension Test	Sample Location
2	19		40% stripping in large aggregates silicius natural sand show stripping, natural aggregates and crushed stone mix	Bottom
	26	91.8	No stripping, poor bondage	Middle
	29	98.0	50% stripping, hot mix with big and dirty aggregates	
	30	81.9	40% stripping, hot mix with big and dirty aggregates	
3	25	38.2	No stripping, very soft sand asphalt	
	27	40.4	No stripping, sand asphalt with low stability	
	28	45.4	No stripping, sand asphalt of very low stability, very soft sand aggregates	
4	10	61.6	5% stripping, crushed stone mix	Bottom
	31	82.8	No stripping, hot mix with fine granulometry	Bottom
	32	80.8	No stripping, good hot mix	Bottom
	33	68.4	No stripping, dense hot mix, high asphalt content	Top
	67	46.0	10% stripping, dense mix with large amount of crushed stones	Surface
5	21	36.5	Stripping in aggregates, dirty aggregates, more granular aggregates in sand asphalt mix	Base
	22	24.7	No stripping, very soft sand asphalt	Base
6	53	76.4	50% stripping, more stripping in sand, fine mix	Surface
	54	72.0	10% stripping, coarse mix	Surface

Table C-5. Results of Tension Test of Dry Samples (contd).

Site No.	Sample No.	Tensile Strength (psi)	Description of Sample After Tension Test	Sample Location
6	60	83.7	50% stripping, fine well graded mix with some natural aggregates	Surface
	61	60.7	10% stripping, coarse mix with some very soft rocks	Surface
	63	47.8	10% stripping, well graded mix, asphalt is little dead looking	Base
7	34	93.8	Hot mix with 15% stripping, dirty aggregates	
	35	110.8	Hot mix with 20% stripping, soft aggregates	
	36	110.0	10% stripping in silicius aggregates, soft asphalt, fine granulometry	Surface
	37	98.5	20% stripping, most stripping in gravel aggregates, good mix	Base
	39	62.9	Some stripping in sand, soft asphalt	Base
	45	83.1	10% stripping in sand, large aggregates in mix, soft asphalt	Base
	46	97.1	15% stripping in crushed silicius aggregates, fine granulometry	Surface
8	1	88.4	30% stripping, crushed stone mix	
	9	61.8	All sample shows stripping and part shows segregation	

Table C-6. Results of Tension Test of Conditioned Samples.

Site No.	Sample No.	Tensile Strength (psi)	Description of Sample After Tension Test	Sample Location
2	17	13.1	Stripping 20% in large aggregates and 90% in natural sand	Top
	18	31.3	Very high stripping, 100%	Bottom
	19	29.9	Stripping in large aggregates, 40%	Bottom
	24	4.3	Stripping 30% in large aggregate and 90% in natural sand	
	68	55.9	20% stripping of aggregates, stripping in natural sand	Surface
	70	94.4	80% stripping in all aggregates, silicius gravel, no crushed stone	Surface
	71	13.6	100% stripping in sand, fine granulometry of asphalt	Base
3	3	88.7	30% stripping in large aggregates, very dense mix with more large aggregates	Top
	4	1.3	Very soft and permeable sand asphalt and aggregate	Bottom
	11	106.6	20% stripping in large aggegates, very dense mix with few large aggregates	
	12	87.1	30% stripping in large aggregates, very dense mix with more large aggregates	Top
	13	8.1	No stripping, very soft sand asphalt, soft aggregates	Bottom
	14	138.5	20% stripping in large aggregates, very dense mix with few large aggregates	
	15	3.8	No stripping, very soft and permeable sand asphalt and aggregate	Bottom
	16	5.1	Same as sample No. 15	Bottom

Table C-6. Results of Tension Test of Conditioned Samples (contd).

Site No.	Sample No.	Tensile Strength (psi)	Description of Sample After Tension Test	Sample Location
4	5	18.7	50% stripping, crushed stone asphalt mix	Bottom
	6	89.8	Very small stripping, crushed stone asphalt mix	Top
	64	51.8	80% stripping, crushed stone aggregate, soft asphalt	Surface
	65	66.7	30% stripping, crushed stone aggregate, good asphalt	Surface
	66	47.1	50% stripping, crushed stone aggregate, good asphalt	Surface
5	20	10.0	Some stripping, sand asphalt with some stones	
	23	11.5	30% stripping, sand asphalt with more aggregates	
6	50	44.9	80% stripping, most stripping in fine aggregates, natural sand, fine granulometry	Surface
	51	39.6	80% stripping in large aggregates, very coarse granulometry.	Surface
	52	28.3	80% stripping in all aggregates, natural sand	Base
	55	89.5	50% stripping in large aggregates, low asphalt content in the mix	Surface
	56	15.2	80% stripping, most stripping is in natural sand	Base
	57	37.0	80% stripping, 100% stripping in natural sand, some crushed stone in the mix	Surface
	58	71.4	50% stripping in large aggregate, coarse mix with rock pieces	Surface
	59	15.1	80% stripping in all aggregates, coarse mix with natural sand	Base

Table C-6. Results of Tension Test of Conditioned Samples (contd)

Site No.	Sample No.	Tensile Strength (psi)	Description of Sample After Tension Test	Sample Location
6	62	30.1	80% stripping in all aggregates, coarse mix with natural sand	Surface
7	7	93.5	Very small stripping, asphalt mix with crushed stone, fine mix	Top
	38	57.8		Base
	40	79.3	60% stripping, fine mix with crushed stones	Surface
	41	71.2	80% stripping, very coarse mix, dead looking asphalt	Base
	42	52.6	90% stripping in all aggregates, good gradation of mix	Base
	43	98.7	60% stripping, crushed stone mix	Surface
	44	62.8	60% stripping, natural sand, poorly graded mix with large rock pieces	Base
	47	61.0	80% stripping in all aggregates, good mix	Base
	48	58.7	50% stripping, fine mix, dead looking asphalt	Base
	49	50.3	60% stripping in all aggregates, natural sand with large stripping percentage, well graded mix	Base
8	2	4.8	50% stripping, 80% stripping in natural sand, very fine mix	
	8	13.9	50% stripping in large aggregates, very high stripping in natural sand	

Table C-7. Results of Boiling Tests

Site No.	Sample No.	Asphalt Retained (%)	Visual Observations	Sample Location
2	18	70-80	All particles coated	Bottom
	26	70-80	Fairly well coated	Middle
	29	70-80	Fairly well coated	
	30	60-80		
3	11	80-90		
	14	80-90		
	25	50-60		
	28	50-60		
4	10	80-90	100% coated original mix	Bottom
	31	70-80		Bottom
	67	70-80		Surface
5	21	80-90	20% particles uncoated	
	22	70-80		
6	54	30-40		Surface
	60	80-90		Surface
	61	50-60		Surface
	63	50-60		Base
7	34	80-90		
	35	70-80		
	44	80-90		Base
	47	80-90		Base
8	1	40-50	Original sample well coated	
	2	40-50	Original sample well coated	
	9	40-50	Original sample well coated	

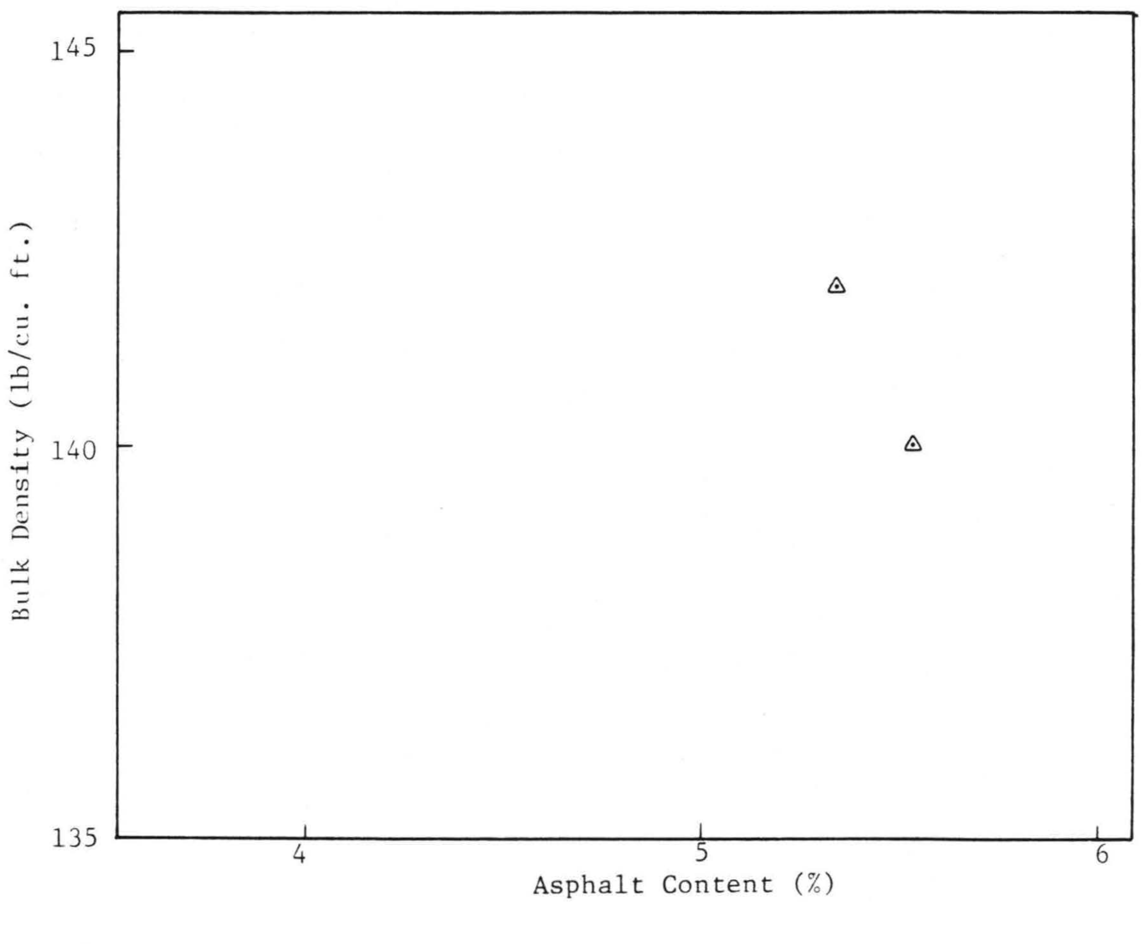


Figure C-1. Relationship between bulk density and asphalt content for surface mixtures of site no. 2.

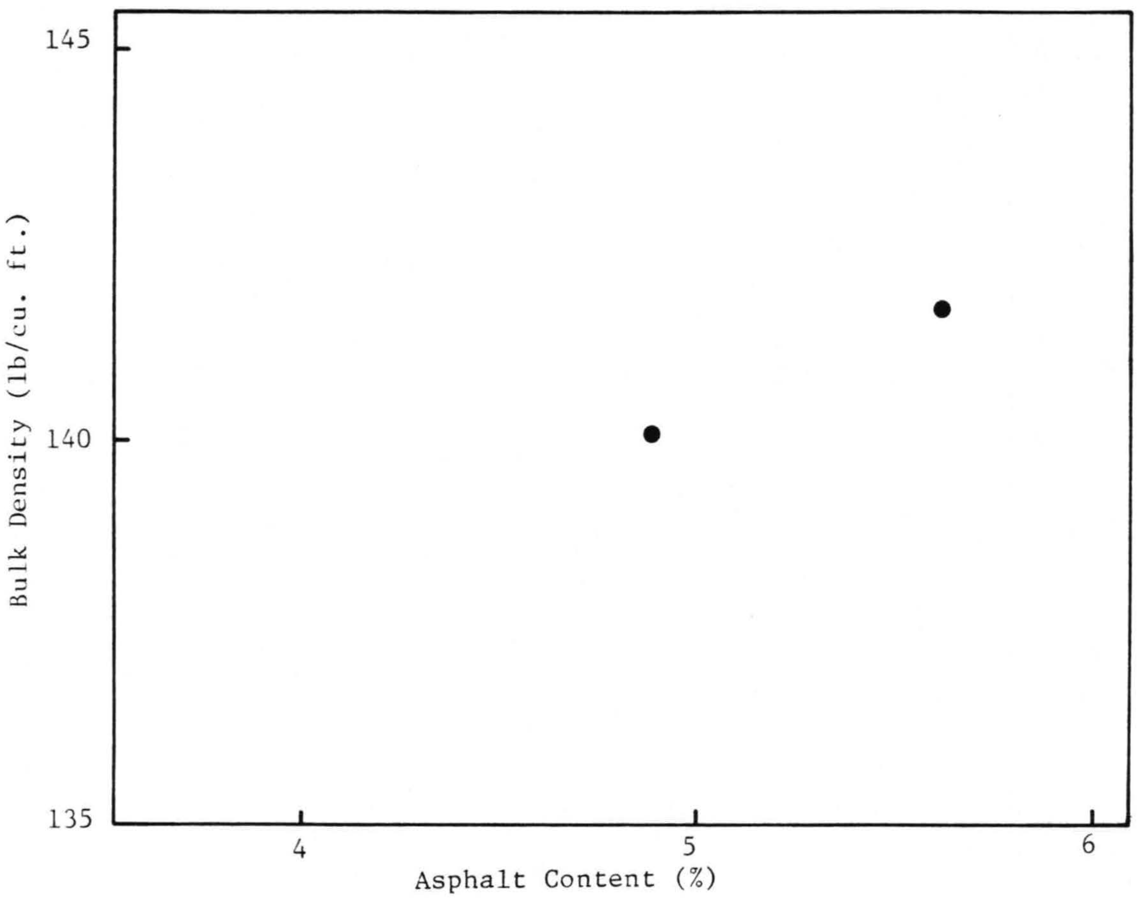


Figure C-2. Relationship between bulk density and asphalt content for surface mixtures of site no. 4.

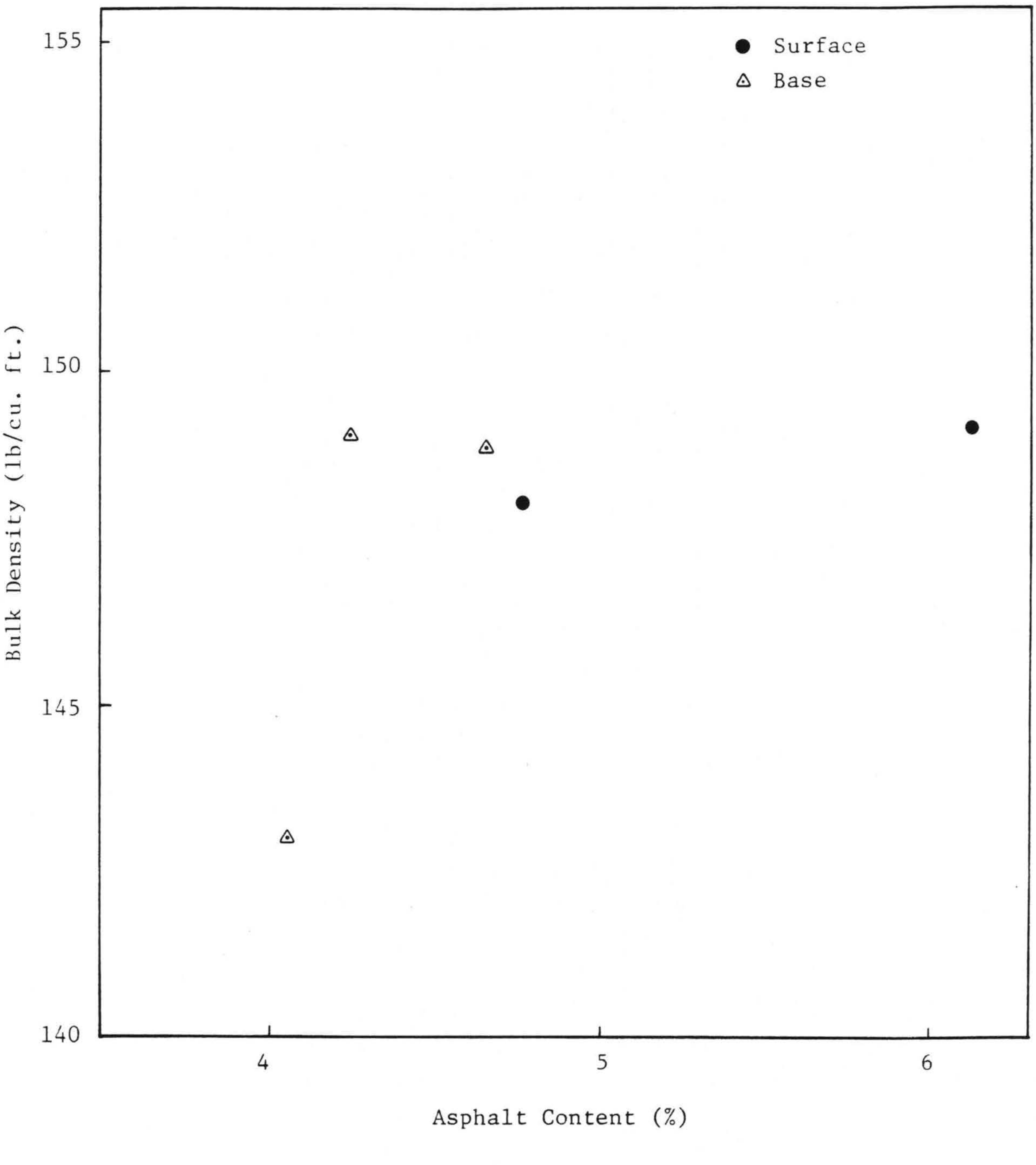


Figure C-3. Relationships between bulk density and asphalt content for surface and base mixtures of site number 6.

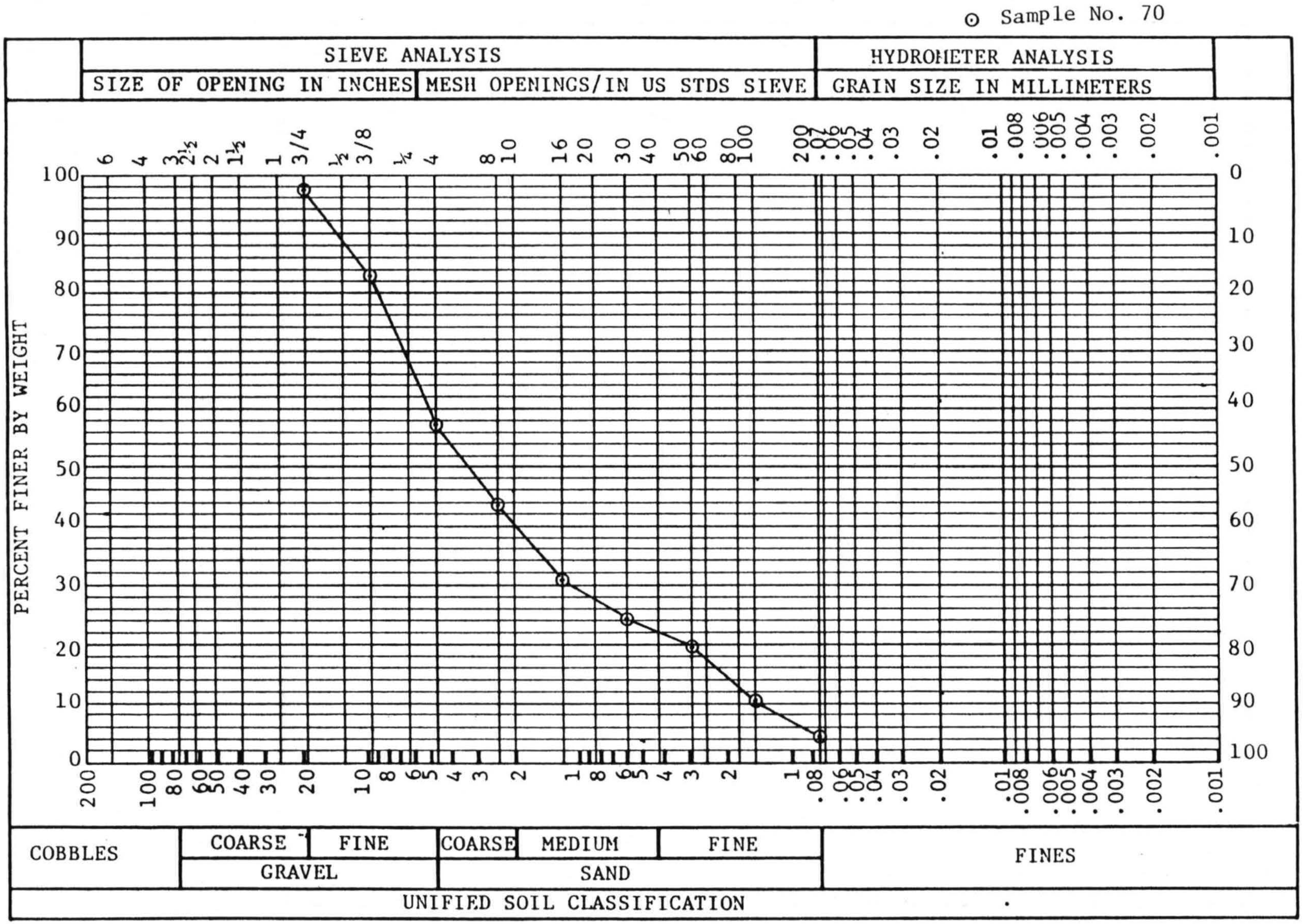


Figure C-4. Particle size distribution of asphalt concrete surface of site No. 2.

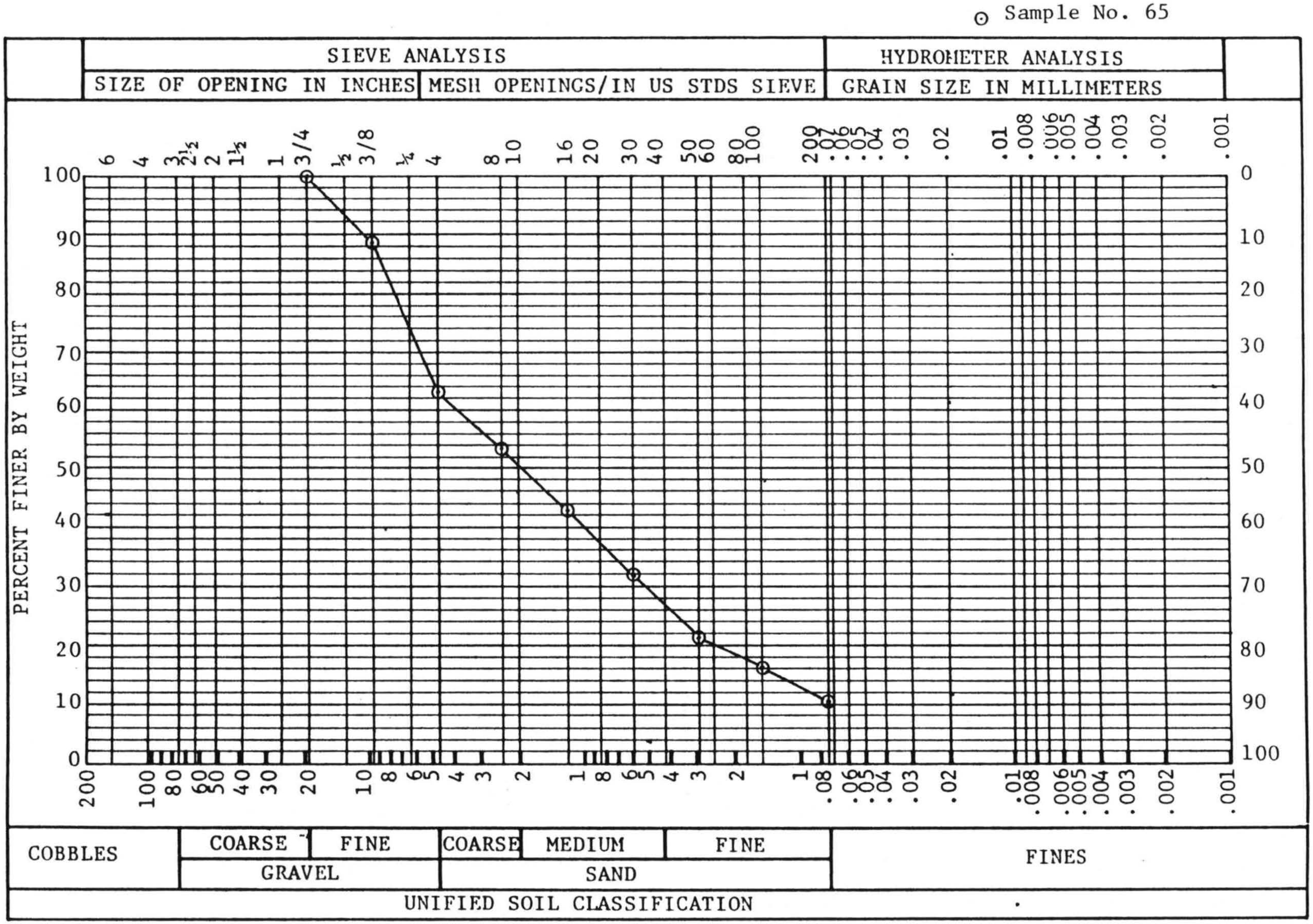


Figure C-5. Particle size distribution of asphalt concrete surface of site No. 4.

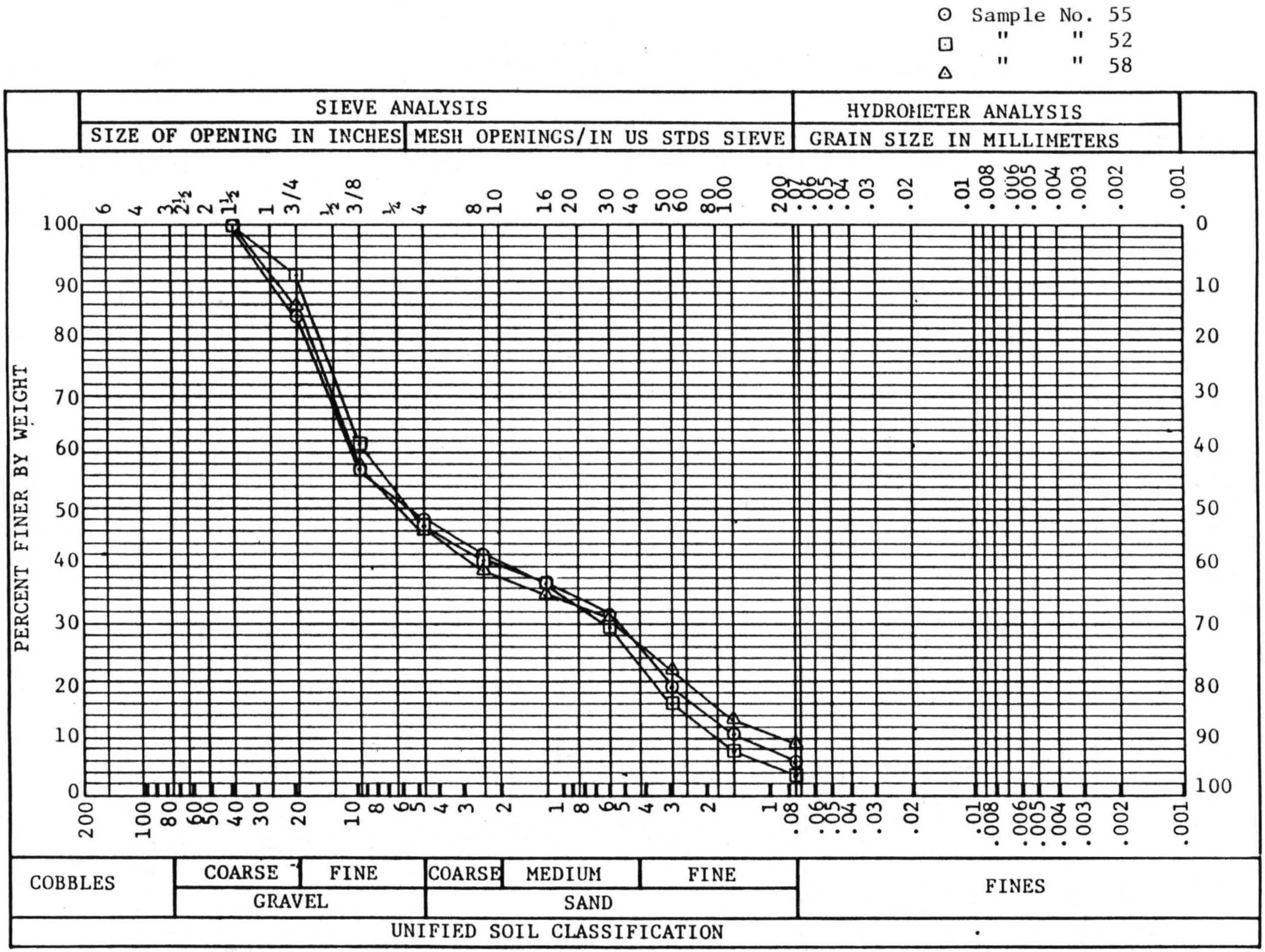


Figure C-6. Particle size distribution of asphalt concrete base of site No. 6.

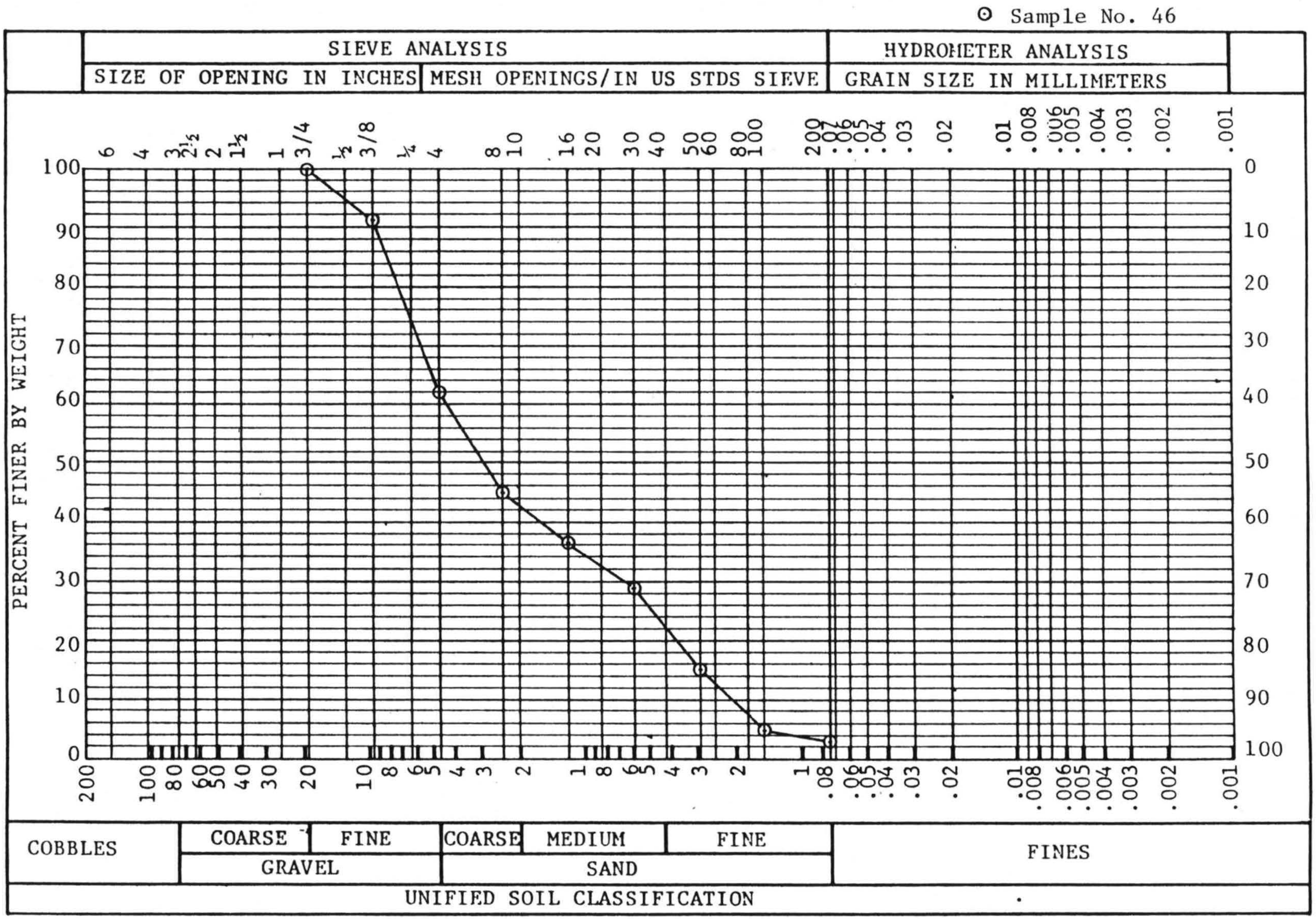


Figure C-7. Particle size distribution of asphalt concrete surface of site No. 7.

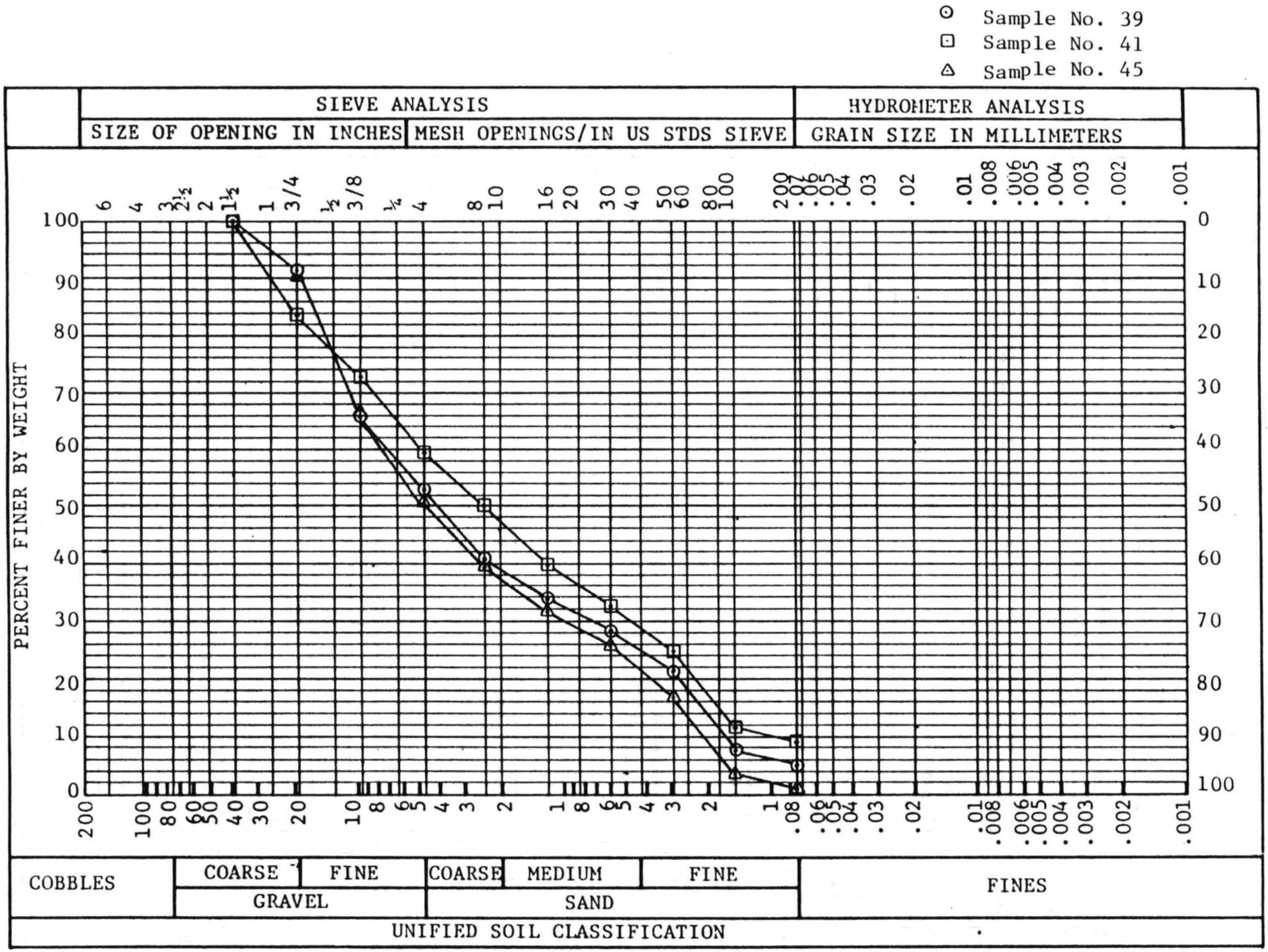


Figure C-8. Particle size distribution of asphalt concrete base of site No. 7.

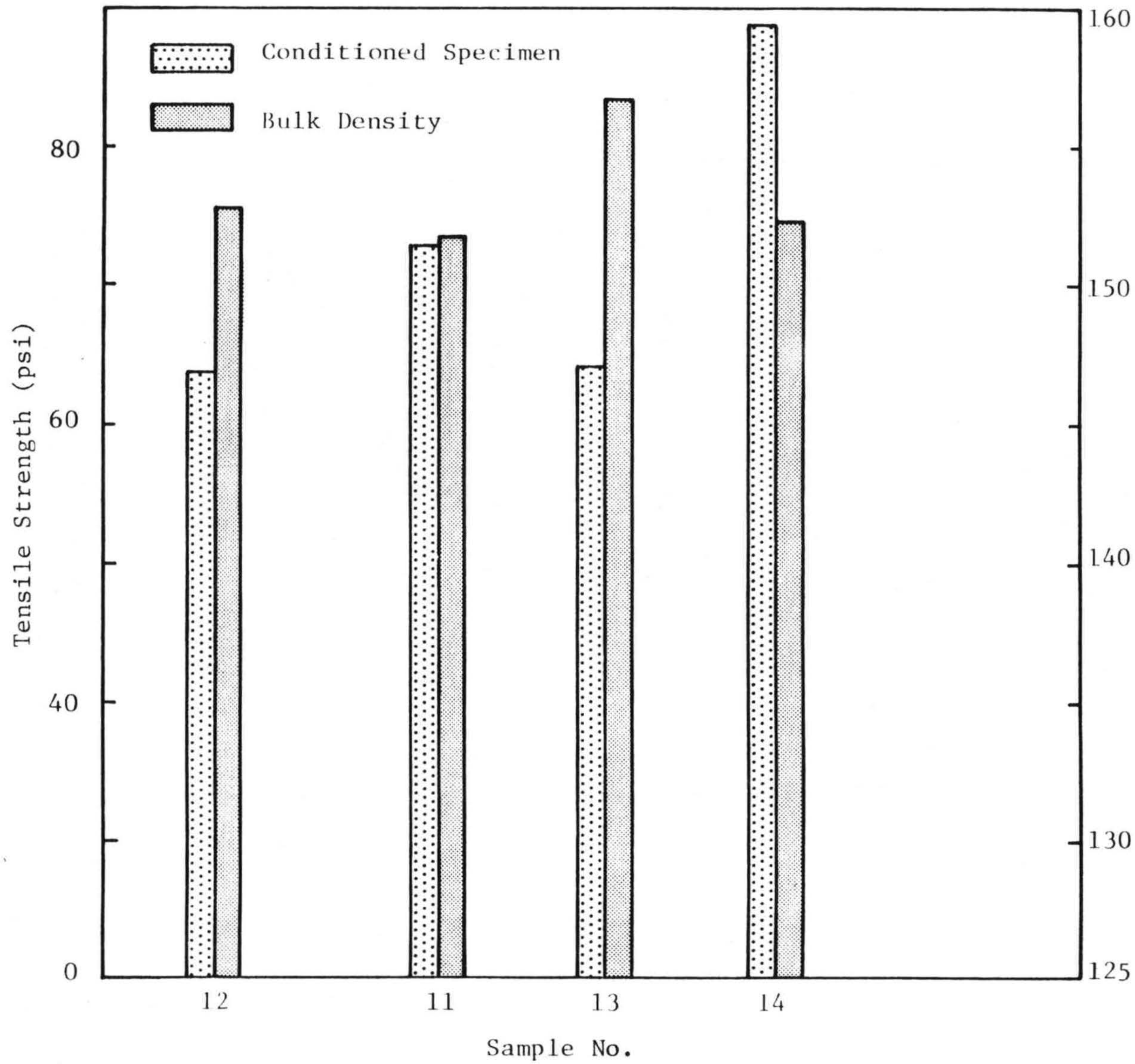


Figure C-9. Tensile strength of conditioned samples and bulk density from specimens from top layers of site no. 3

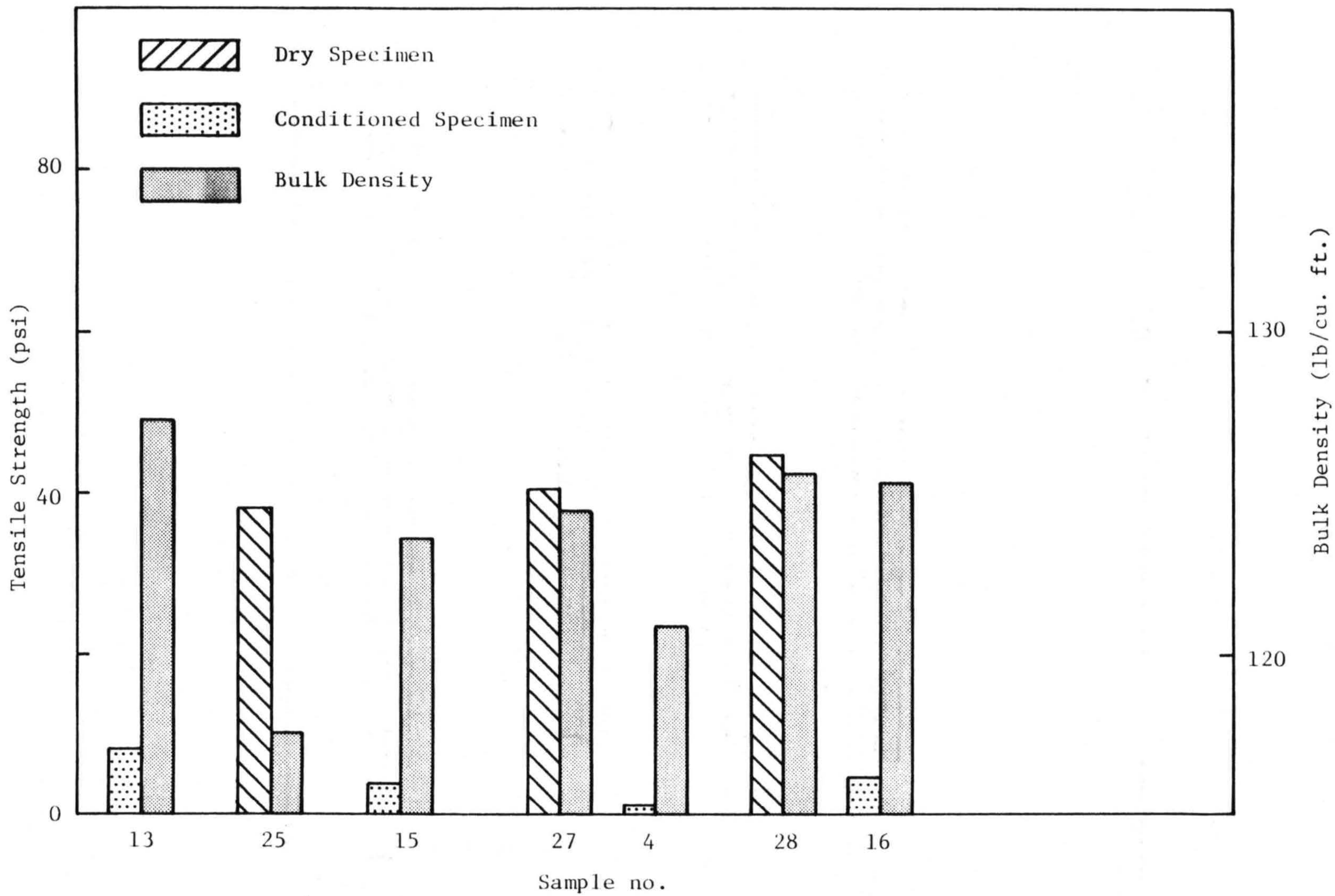


Figure C-10. Tensile strength for dry and conditioned samples along with bulk density for site no. 3.

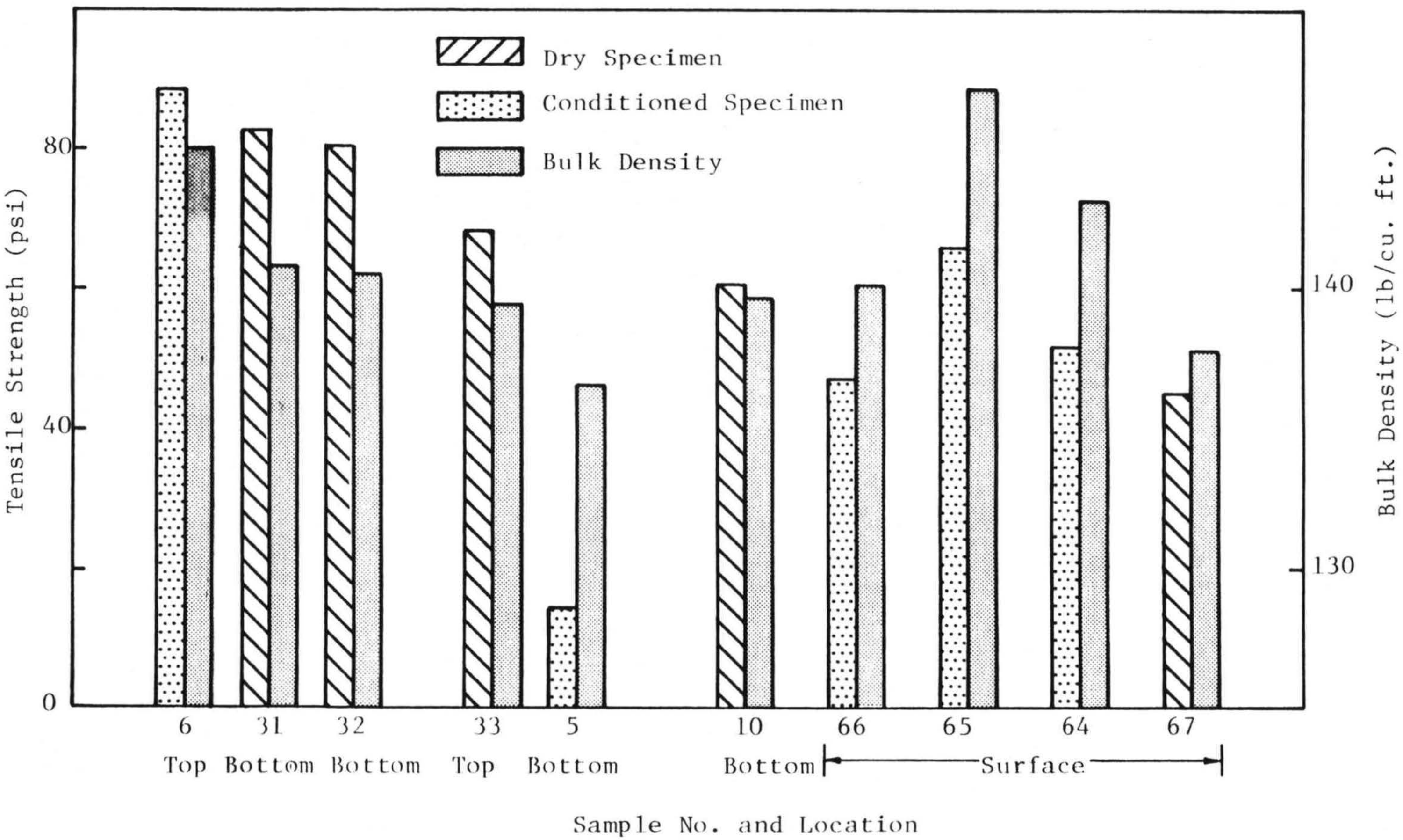


Figure C-11. Tensile strength of dry and conditoned samples along with bulk density for site No. 4.

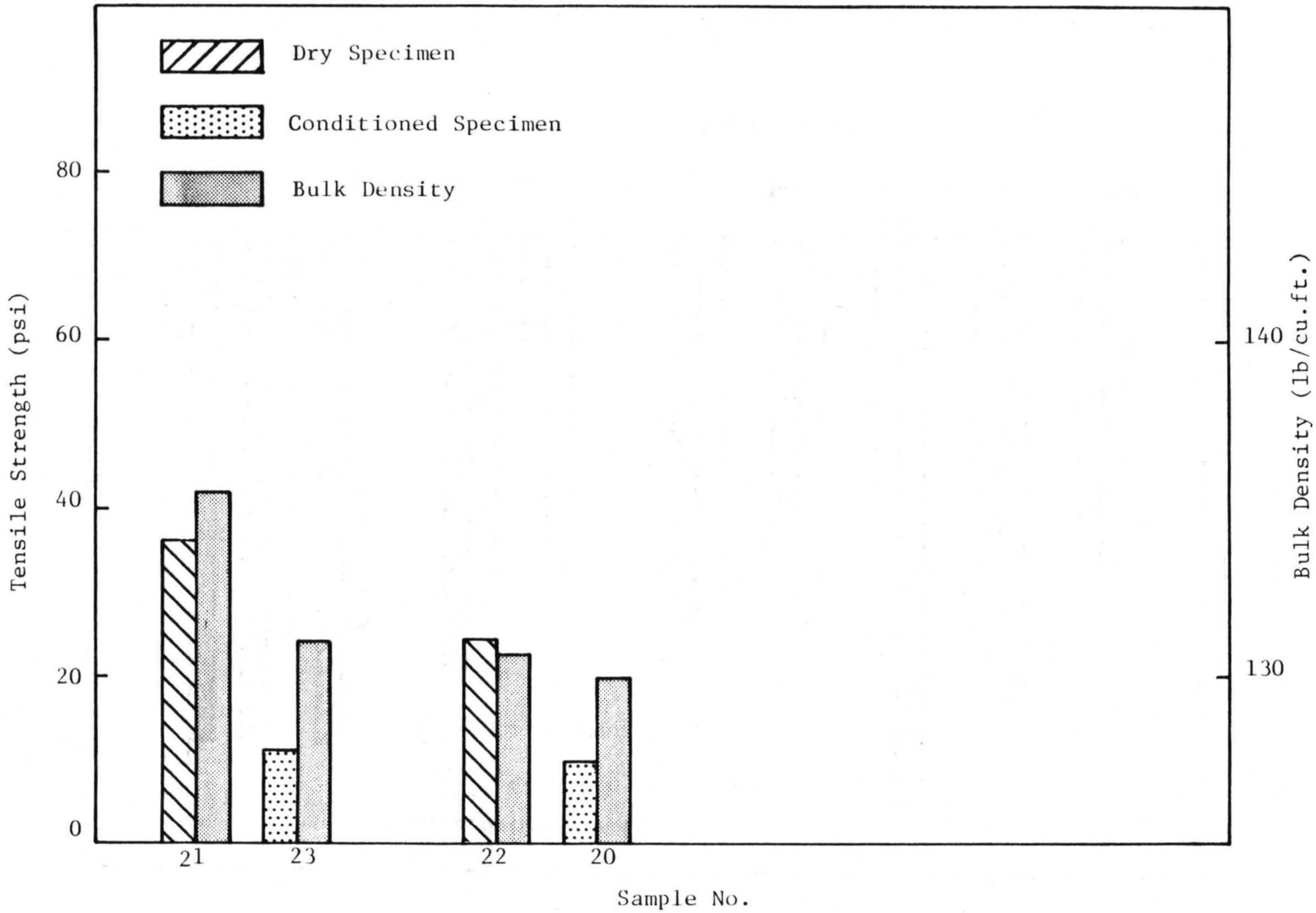


Figure C-12. Tensile strength for dry and conditioned samples along with bulk density for site no.5.

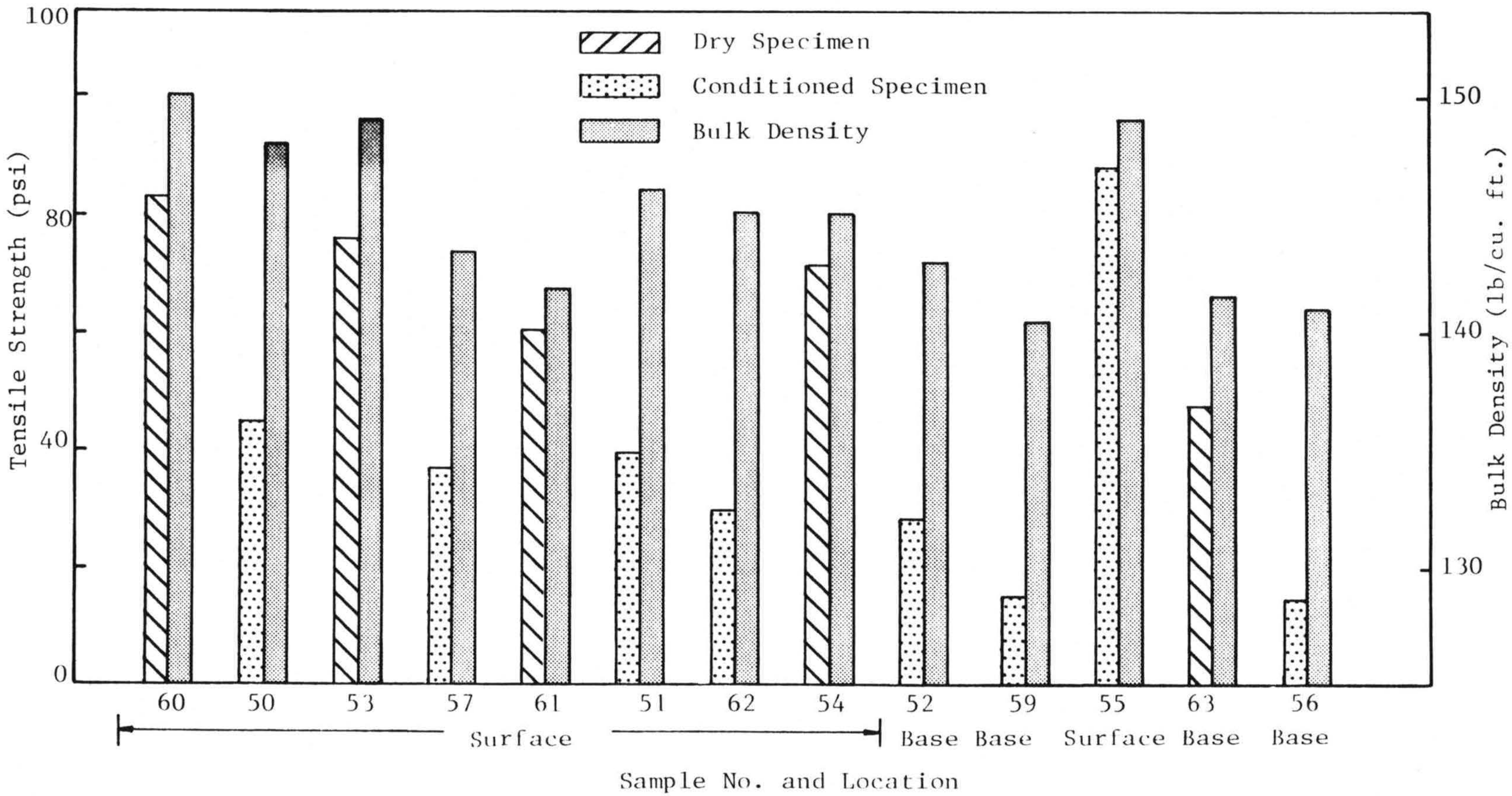


Figure C-13. Tensile strength for dry and conditioned samples along with bulk density for surface and base layers of site No. 6.

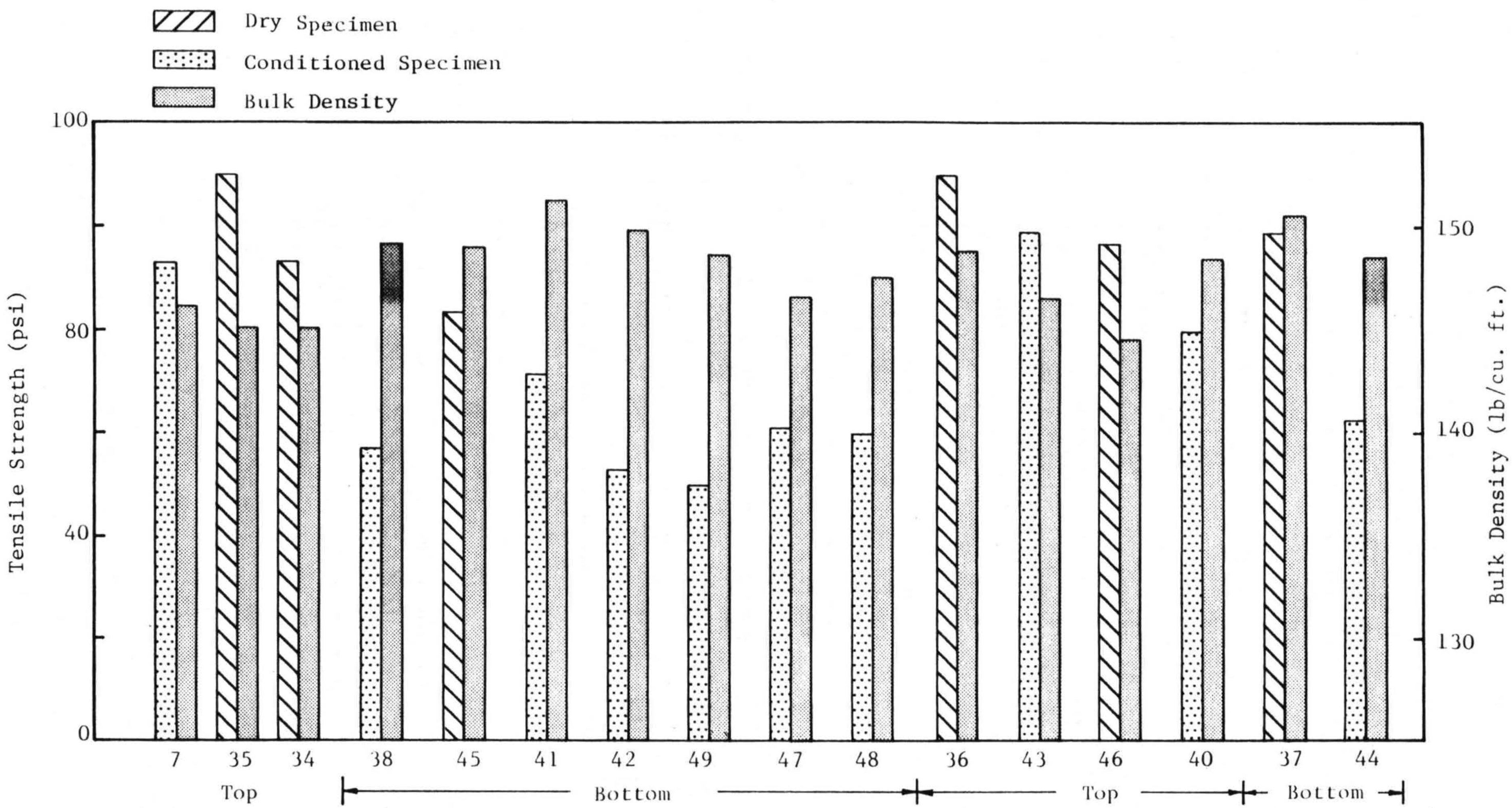


Figure C-14. Tensile strength for dry and conditioned samples along with bulk density for surface and base layers fo site No. 7.

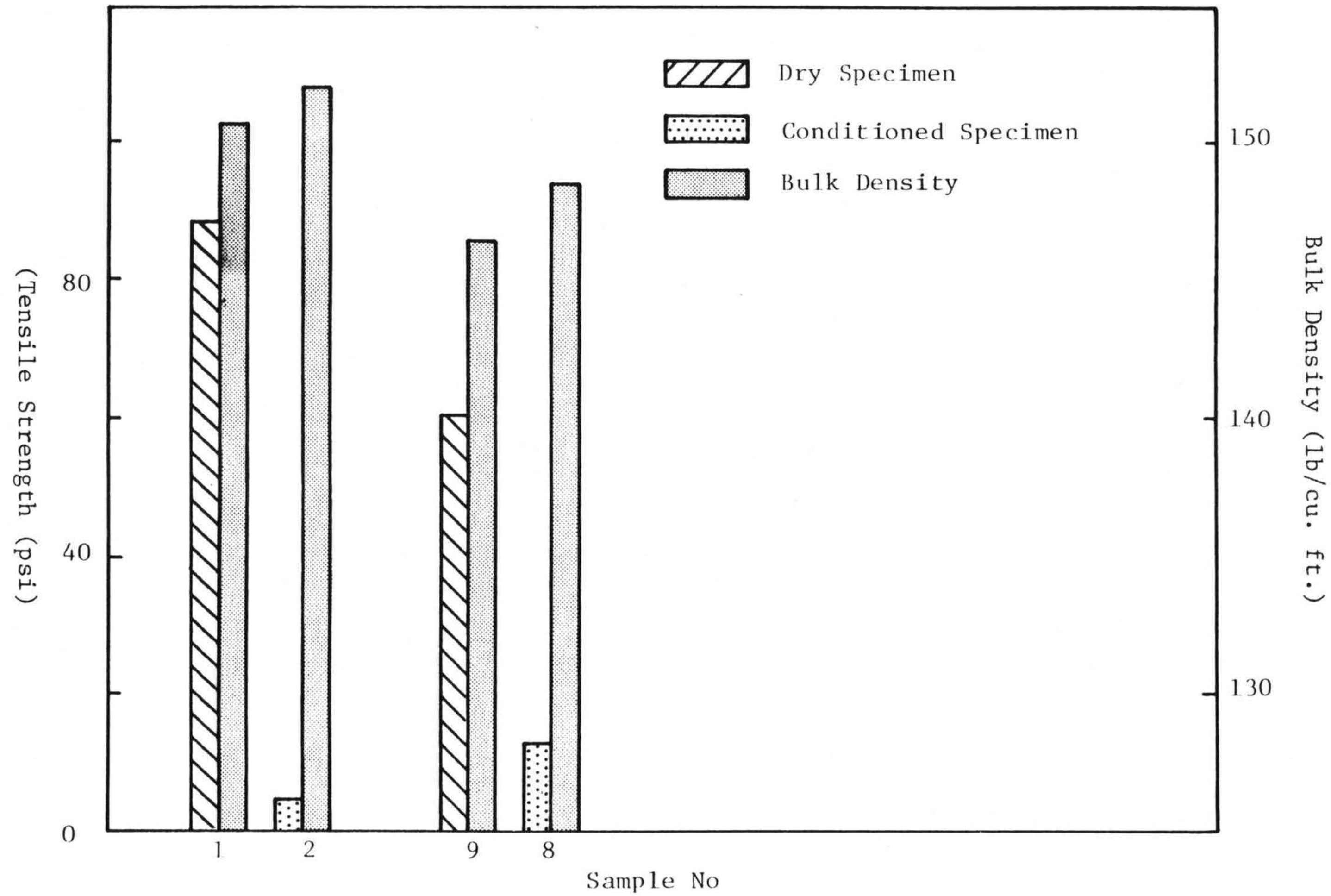


Figure C-15. Tensile strength for dry and conditioned samples along with bulk density for site no. 8.

APPENDIX D

METHOD AND OPERATING CHARACTERISTICS OF NDT DEVICES

A brief description of the Benkleman Beam, Dynaflect, and FWD are presented in this section

Benkleman Beam

A Benkleman beam is a surface deflection-measuring device that operates on a simple lever-arm principle and uses a dial mechanism for deflection measurements. The tests are usually conducted at creep speed of the test vehicle. The test vehicle consists of a two-axle, six-tire dump truck with a specified load on the rear axle. The tip of the Benkleman beam is placed on the road surface just ahead and between the rear set of dual tires. As the test vehicle rolls past the tip, maximum deflections are measured by the dial mechanisms. To obtain an estimate of Young's moduli of pavement layers, a deflection basin or bowl is needed where deflections are measured at a couple of points away from the load as well as under the load. Dynamic NDT devices such as the Dynaflect has been a favorite during the last two decades.

Dynaflect

The Dynaflect is a small trailer towed behind a pickup truck. In the operating mode, the Dynaflect applies an oscillating load at a frequency of 8 Hz on the pavement with a peak-to-peak amplitude of 1000 pounds. The deflection of the pavement is measured by a series of five sensors. These sensors are arranged to measure the peak-to-peak surface deflection midway between the loading wheels (Sensor 1) and at one-foot intervals away from Sensor 1. Figure D.1 illustrates the sensors' configuration. The electronics of the system provide a readout of the deflection in milli-inches. These deflection data are then used to evaluate the structural capacity of the roadway. The Dynaflect is nondestructive, causing no damage to the pavement, and is completely mobile allowing for a fast production of deflection measurements. Figure D.2 illustrates a typical Dynaflect deflection basin.

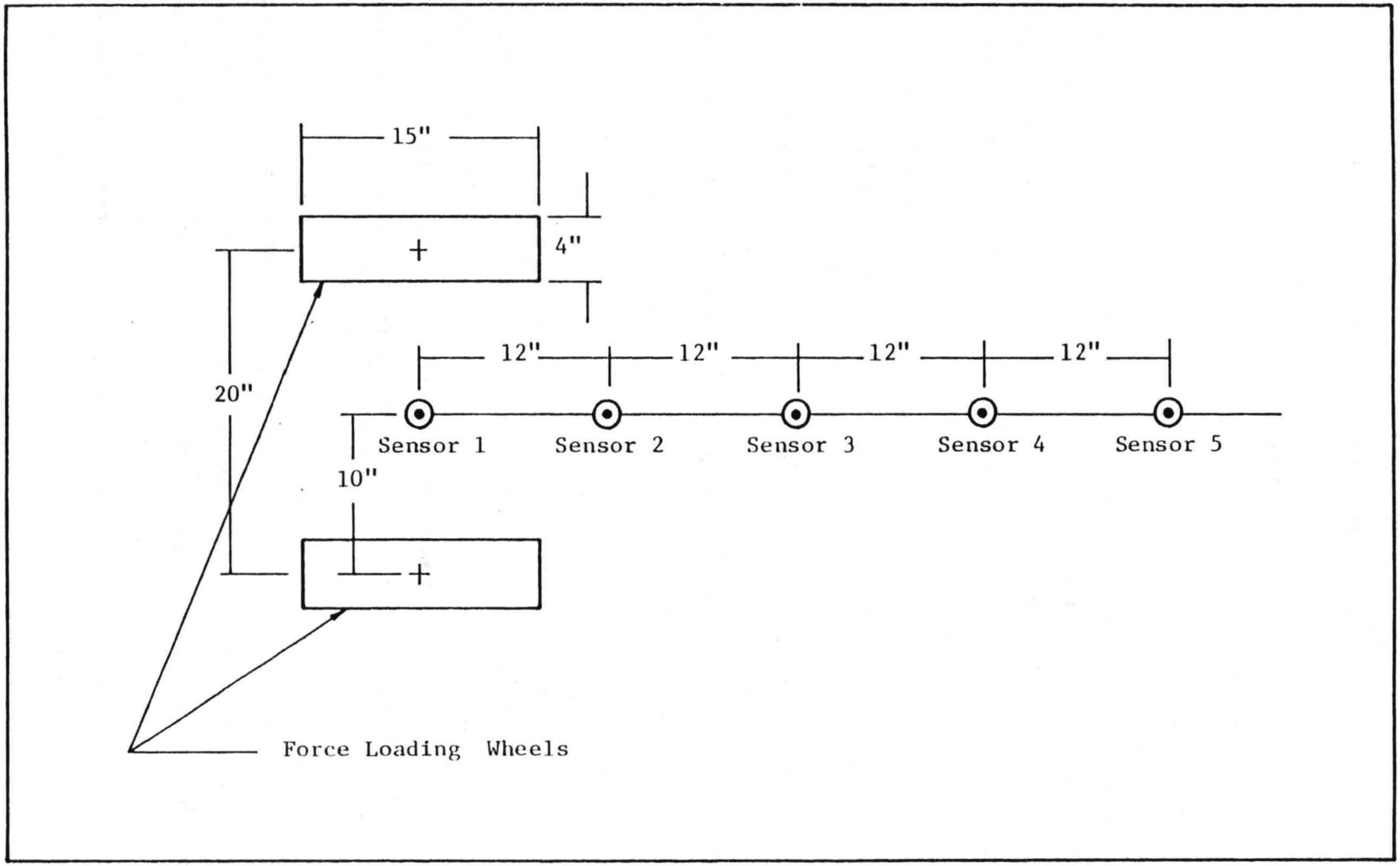
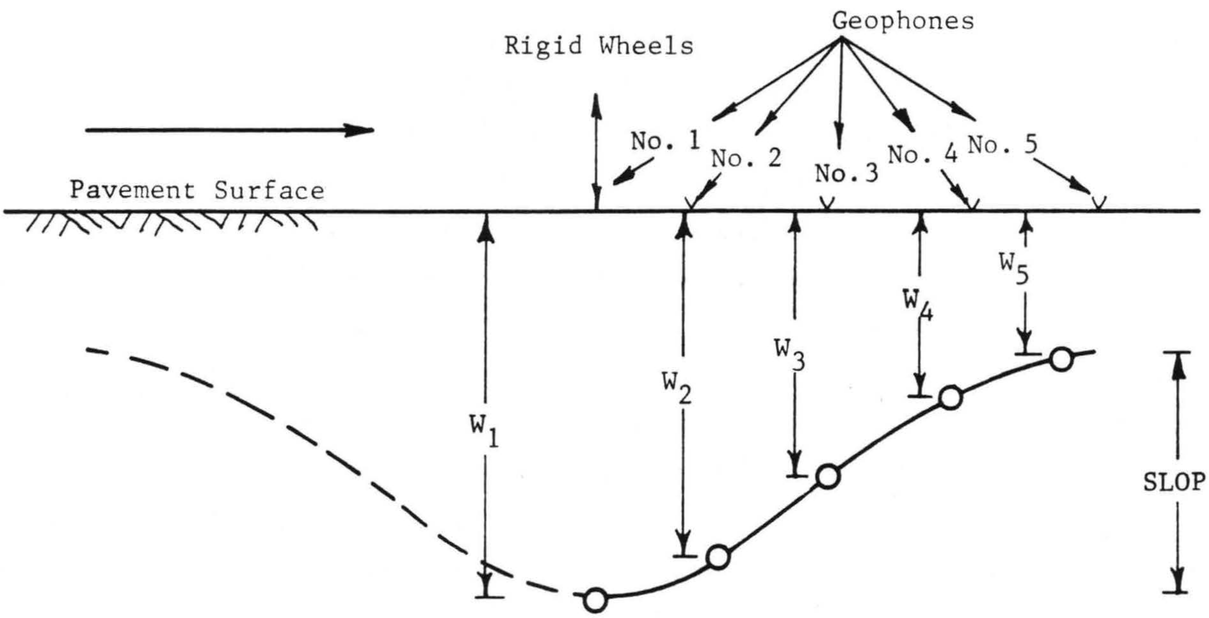


Figure D-1. Dynaflect measuring array.



Maximum Dynaflect Deflection = W_1

W_1 to W_5 are deflection readings.

Basin Slope - SLOP = $W_1 - W_5$

Figure D-2. Typical Dynaflect deflection basin.

Falling Weight Deflectometer

A Falling Weight Deflectometer (FWD) applies an impulse load by dropping a known mass from a predetermined height as illustrated in Figure D.3. In the present study, a recent model of DYNATEST's Falling Weight Deflectometer was used. The FWD is a trailer-mounted device which can be towed by any standard passenger car or van at highway speed. The transient pulse-generating device is the trailer-mounted frame capable of directing different sets of mass configurations to fall from a preset height perpendicular to the surface. This allows the capability of producing a wide range of peak-force amplitudes by varying mass and/or height. The assembly consists of mass, frame, loading plates, and a rubber buffer which acts as a spring. The operation of lifting and dropping a mass on the loading plate is based on the electro-hydraulic system. The falling weight/buffer sub-assembly is furnished such that different configurations of mass may be employed. The FWD produces a transient reproducible load pulse approximately half-sine wave formed and 25 to 30 milliseconds in duration. For routine testing, a loading plate of 11.8 inches (300 mm) in diameter is used. The mass guide shaft is perpendicular to the road surface in the measuring mode as well as the transport mode. The system includes a load cell which is capable of measuring the peak force that is applied perpendicular to the loading plate.

The system provides at least seven separate deflection measurements per test. One of the deflection-sensing transducers (geophones) measures the peak deflection of the pavement surface at the center of the loading plate, while the six remaining transducers are capable of being positioned along the raise/lower bar at varying distances from the center of the loading plate. The unit is capable of testing in the long-distance towing position by simply lowering the loading plate/mass/seismic detector bar subassembly to the pavement surface from controls located within the towing vehicle.

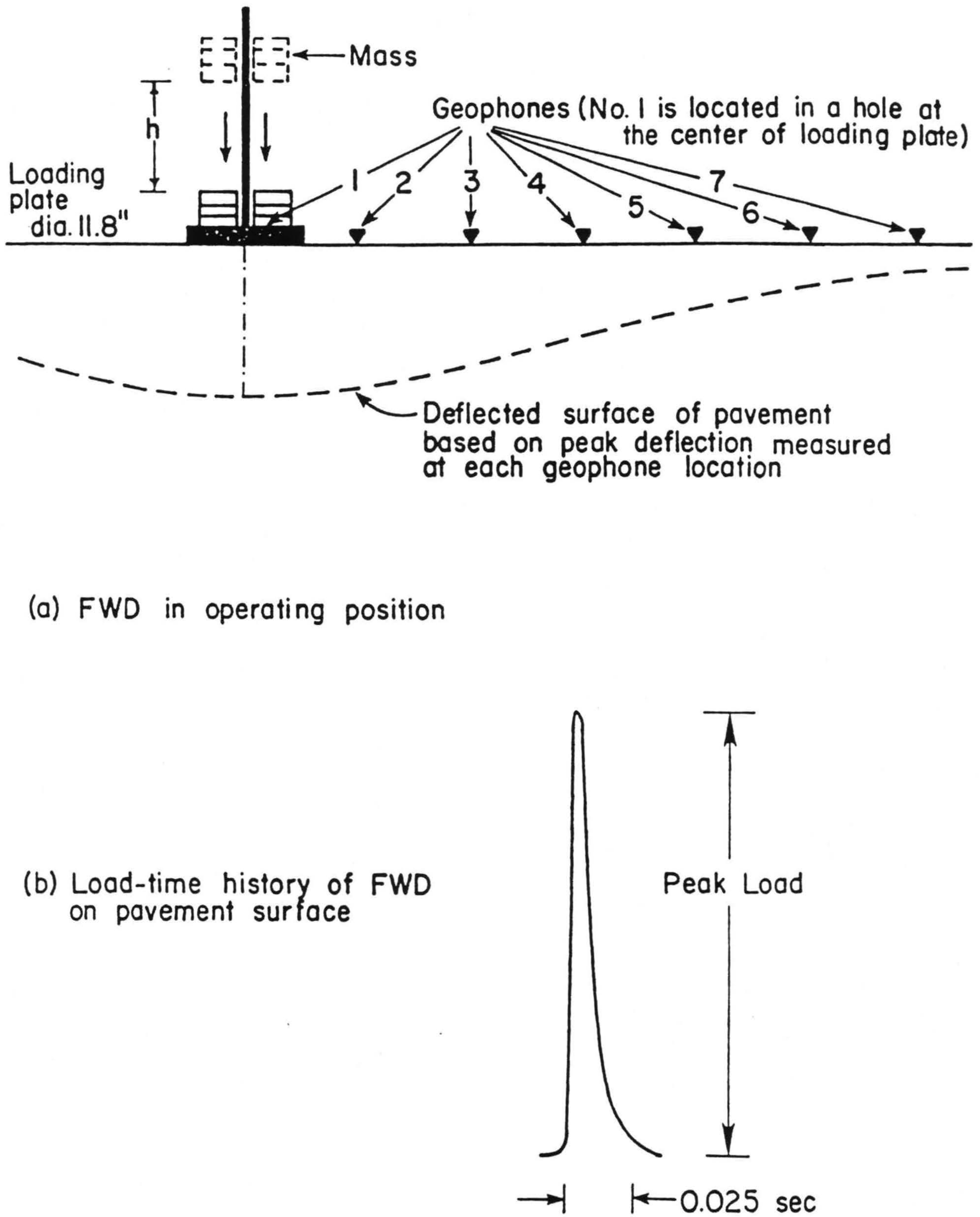


Figure D-3. Principle of a Falling Weight Deflectometer, FWD test (Ref 5).

APPENDIX E
ANALYSIS OF DYNAMIC DEFLECTION BASINS

For the mechanistic evaluation of the deflection data, deflection basins are needed in addition to the maximum deflection. Therefore, dynamic deflection basins measured by the Dynaflect and the FWD were considered in this study for analysis.

Background

Traditionally, pavement evaluation and overlay design have been based on limiting deflection criteria and empirical relationships developed from field studies of maximum deflection and pavement performance. Overlay thickness requirements are determined from nomographs developed from these empirical relationships which can reduce deflection below the limiting deflection criterion. Correlation studies made with other NDT devices such as the Dynaflect enables the use of these nomographs. These empirical methods are based solely on local experience, and, therefore useful for limited applications. Maximum deflection is indicative of total pavement response and, alone, it cannot lend to the evaluation of the structural integrity and material characterization of different pavement layers. It can be shown that two different pavements show the same maximum deflection value but can have different Young's moduli of layers if the deflection basins exhibit different shapes. Deflection basins measured by NDT devices such as the Dynaflect, Road Rater, and FWD have been characterized by different parameters which are functions of deflection values at one or more sensors (Figure D.2). Various deflection basin parameters (Figure D.2) have been correlated to pavement moduli based on layered theory computations. Graphical- and nomograph-based procedures using basin parameters have been developed generally for a two-or three-layers pavement model.

All the graphical procedures based on deflection basin parameters are of limited use because of the following reasons.

(1) These are developed for a specific NDT device.

(2) Layered theory or any other structural model's computations used to develop these procedures are based on specific ranges of moduli of pavement layers. This factor is often neglected when a user applies these type of nomographs to practice.

(3) These are limited to two or three layers and for a particular pavement type.

(4) In general, the bottom layer is assumed to be semi infinite which can result in a large over estimation error in the subgrade modulus if a rock layer exists within 20 feet from the pavement (Ref 4).

Mechanistic Models for NDT Evaluation

The background and specific procedures for mechanistic analyses of deflection basins are presented in the following sections (Ref 5).

Methodology. The most widely used analytical procedures for mechanistic interpretation of deflection basins measured on pavements are based on multilayered linear elastic theory. As shown in Figure E-1, the layered model of an existing pavement can be used for in situ characterization of materials in each layer. Later, this information can be used again in the layered theory computations to estimate its load carrying capacity and for overlay design. Application of layered theory for in situ material characterization from a deflection basin requires estimation of only one unknown parameter, Young's modulus of elasticity (E) of each layer. Poisson's ratio can be assumed from literature without any significant effect on pavement response due to small variations in its

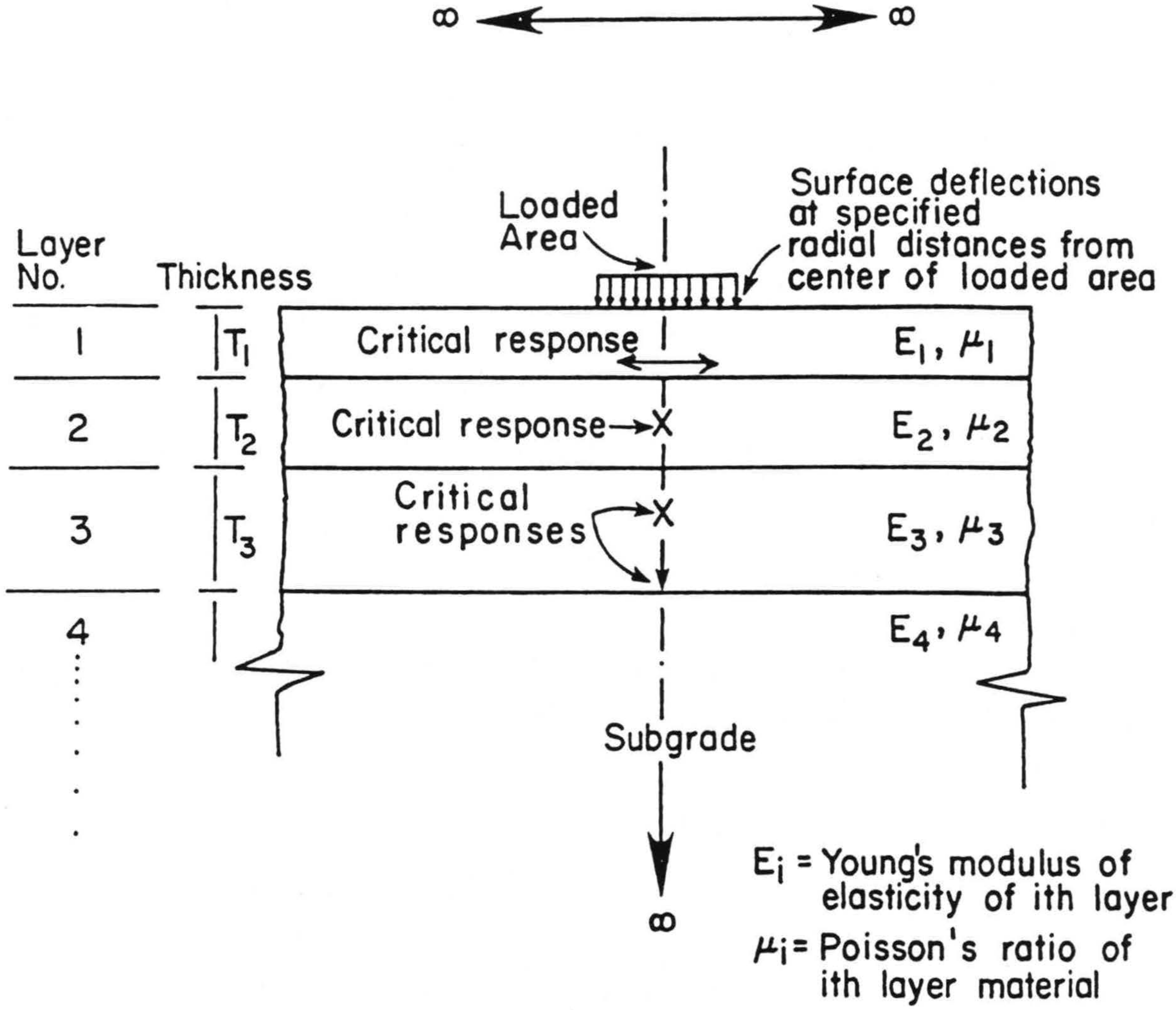


Figure E-1. An idealized multilayer linearly elastic pavement (Ref 5).

values. Following are some important assumptions made in layered theory solutions:

(1) Material in each layer is linearly elastic, homogeneous, and isotropic.

(2) Each layer above the elastic half-space is considered weightless, and finite in thickness but assumed to be infinite in the horizontal plane.

(3) A uniform static load is applied on a circular area of the surface.

(4) The effect of inertia is neglected.

Layered theory solutions are based on axisymmetric condition; therefore, the principle of superposition is applied to determine the effect of more than one load.

In recent years, researchers have used an iterative procedure of applying layered theory in reverse order by changing modulus value in each iteration until a best fit of predicted and measured basin is obtained. The moduli in the best-fit iteration represents the in situ moduli. This approach is very promising as it can be applied to a multilayered flexible or rigid pavement. Uddin, et. al. (Ref 6) used this approach to determine in situ moduli of rigid pavements considering a subgrade of semi-infinite, as well as finite, thickness.

Dynamic deflection basins measured for the purpose of in situ material characterization are analyzed by inverse application of elastic layered theory to derive in situ Young's moduli of pavement layers which is the first step of the structural evaluation. The second step is to correct modulus of the pavement material which is temperature sensitive or exhibits nonlinear behavior. The majority of the existing evaluation procedures stop here and further application of the derived moduli is left to the user's discretion. The self-iterative computer programs used in this study feature additional analyses for (1) the calculation of critical stress or strain, (2) the estimation of fatigue life using the critical response, and (3) the determination of the remaining life of the pavement. Tabulated results of remaining life and pavement moduli with distance along the pavement can then be used to identify areas which need an overlay and to calculate design moduli value. The in situ Young's moduli can then be used for overlay design using mechanistic procedures of overlay design. An important concept used in this study is to treat every deflection basin on an individual basis for the analysis. The two computer programs, RPEDD1 and FPEDD1, have been developed recently at the University of Texas at Austin (Ref 5). A simplified flow chart of the program RPEDD1 is shown in Figure E-2. Both of these programs use ELSYM5 (Ref 7) for computation of the theoretical pavement response.

Description of RPEDD1 and FPEDD1. Some important concepts and features related to RPEDD1 and FPEDD1 are presented in the following section. The computer program RPEDD1 (A Rigid Pavement Structural Evaluation System Based on Dynamic Deflections) and FPEDD1 (A Flexible Pavement Structural Evaluation System Based on dynamic Deflections) use the basic approach of fitting a dynamic deflection basin by applying successive corrections in the initially assumed moduli and layered theory computations to derive in situ Young's moduli. These self iterative programs are based on certain assumptions related to input parameters and output response, establishing tolerances in deflections, moduli, criterion of acceptable limits for moduli, and consideration of a finite thickness of subgrade. The simplified assumptions necessary to validate the application of layered theory for determining in situ moduli from a deflection basin can be separated into two groups. The first group consists of assumptions inherent with the use of layered linear elastic theory to calculate pavement structural response. These are related to material properties, thickness information, and boundary conditions as described earlier. The assumptions of the second group are listed below.

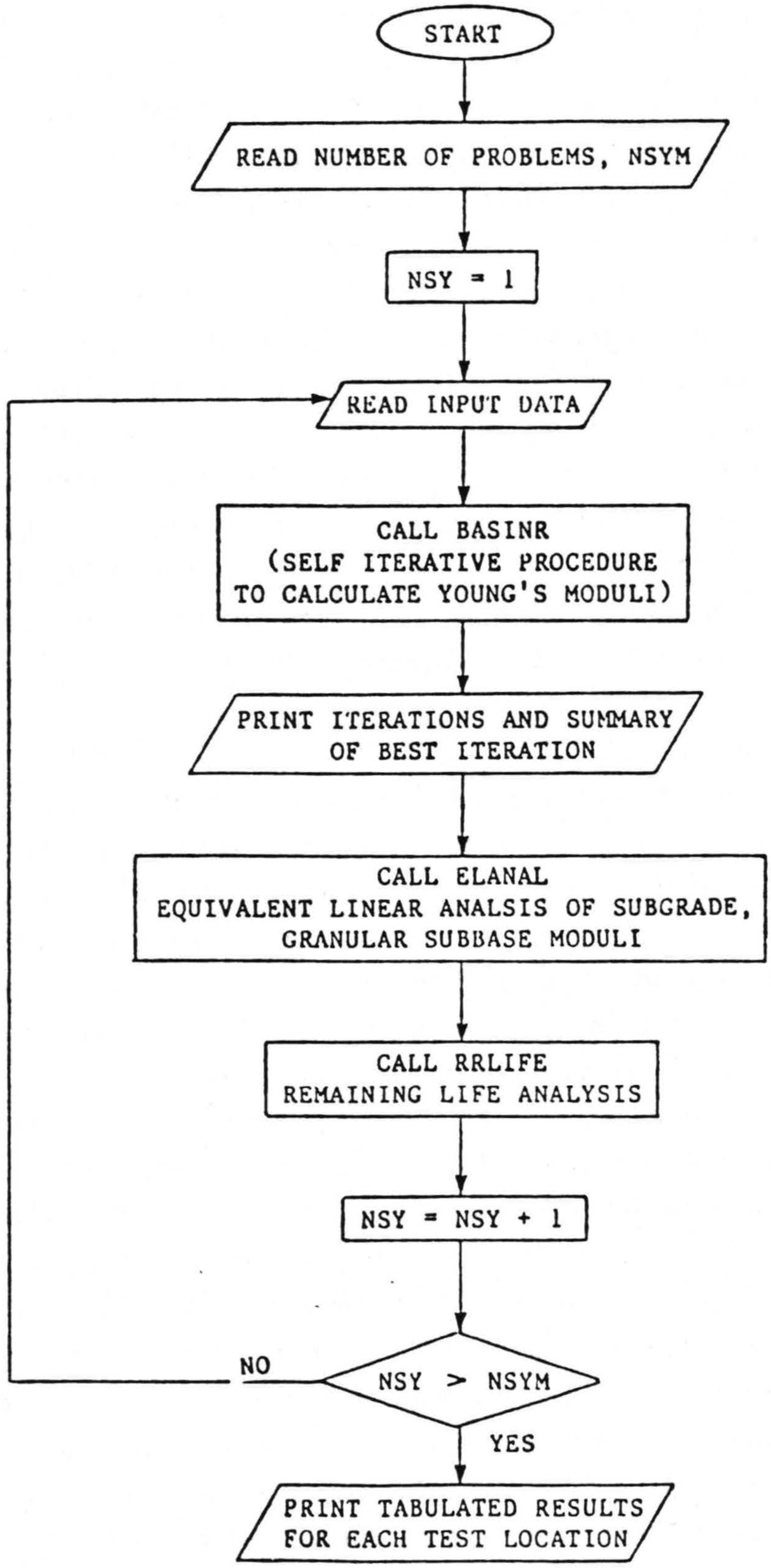


Figure E-2. Simplified flow chart of RPEDD1 (Ref 5).

(1) The existing pavement is considered to be a layered linearly elastic system. Therefore, the principle of superposition is valid to calculate pavement response due to more than one load.

(2) The peak-to-peak dynamic force of 1000 lbs of the Dynaflect is modelled as two pseudo static loads of 500 lbs each uniformly distributed on circular areas (167 lbs/sq in. on each circular area). The peak dynamic force of the FWD is assumed to be equal to a pseudo static load uniformly distributed on a circular area with a radius of 5.9 inches (e.g., radius of the FWD loading plate).

(3) The thickness of each layer is assumed to be known. All layers are assumed to be in perfect contact, parallel to each other and extend to infinity in horizontal directions. For rigid pavements, the deflection basin is measured by loading midspan between the joints or transverse cracks and far from the pavement edge to satisfy this assumption.

(4) The subgrade is characterized by assigning an average value to its modulus of elasticity.

(5) The temperature effect is neglected.

The self-iterative methodology relies on generating theoretical deflection basins using ELSYM5 and changing the initial values of assumed moduli through a procedure of successive correction in order to obtain a best fit of the measured deflection basin. The discrepancies in the theoretical and measured deflections have been related to the required corrections in the preceding values of moduli. The correction procedure is designed to handle deflection basins of both the Dynaflect and FWD. A conceptual treatment of the procedure of successive correction is presented below.

$$ERR_j = DEFM_j - DEF_j \quad (E.1)$$

and

$$ERRP_j = 100\ (ERR_j/DEFM_j) \quad (E.2)$$

where

Subscript j refers to deflection sensors (j = 1 to 5 for Dynaflect; j = 1 to 7 for FWD).

$DEFM_j$ = deflection measured at j^{th} sensor.

DEF_j = deflection calculated at j^{th} sensor.

ERR_j = error, difference between measured and calculated deflection at j^{th} sensor.

$ERRP_j$ = percent error in measured and calculated deflections.

To start with, deflections are calculated from the initial input values of moduli, referred to as seed moduli in this study. Number of iterations in the first cycle is equal to the number of layers in the pavement. In each cycle, the first iteration is made to correct subgrade modulus. ELSYM5 is then called to calculate theoretical deflections. The procedure of successive corrections to the modulus of the next upper layer and use of ELSYM5 to calculate theoretical deflections is continued until the moduli of all layers have been checked for correction. Then another cycle starts again from the subgrade layer. The relationship used in the procedure of successive correction is given below in the generalized form.

$$ENEW_i = E_i\ (1.0 - CORR_i \times ERRP_k \times 0.5) \quad (E.3)$$

where

$ENEW_i$ = corrected value of Young's moduli of i^{th} layer,

E_i = value of Young's modulus of i^{th} layer in the previous iteration (in the first iteration, it is the seed modulus),

$CORR_i$ = correction factor (for i^{th} layer) applied to the discrepancy in measured deflection and calculated deflection, and

$ERRP_k$ = discrepancy in calculated (based on E_i's) and measured deflections of k^{th} sensor(s) in terms of percent error as calculated in Equation E.2.

By applying appropriate correction to the corresponding modulus values, only half of the discrepancy in the measured and calculated deflection is aimed to be removed. A number of additional measures are implemented in the self-iterative models to ensure efficiency, reliability and accuracy of the finally-derived moduli which are discussed in the next section. Iterations are discontinued whenever one of the following criteria is reached first.

(1) The maximum absolute discrepancy among calculated and measured deflections is less than or equal to the permissible tolerance.

(2) Whenever any correction in a modulus value causes the discrepancies in calculated and measured deflections to increase. It is an important criterion to ensure that the solution is not going in the wrong direction.

(3) The allowable maximum number of iterations is reached.

A simplified flow diagram of the computer program RPEDD1 for determining in situ Young's moduli from a deflection basin is illustrated in Figure E.2. In the case of FPEDD1, a similar flow diagram has been used with the addition of a temperature correction for the surface asphaltic concrete modulus after the evaluation of in situ moduli and computation of critical response under design load. Both programs are designed to handle a 3- or 4-layer pavement. The default maximum number of iterations in RPEDD1 and FPEDD1 is 10. The experience gained in using the programs indicates that generally a solution is reached in less than 10 iterations if the seed moduli are not drastically far from actual values or if the option of default seed moduli is used. If the initial seed moduli are very close to the actual value, then generally a unique set of Young's moduli can be easily reached within a reasonable margin of error. The approach used in these programs is to develop relationships which can be used to predict seed moduli from measured deflections. Therefore, any guess work in seed moduli is eliminated. Furthermore if only one unique set of moduli is generated by the program internally using other input data, then the derived moduli using the self-iterative model will also be unique within an acceptable margin of error. Several predictive relationships for seed moduli are used in the default procedures of the self-iterative models contained in RPEDD1 and FPEDD1.

All asphalt-bond materials in pavements exhibit temperature sensitivity. The asphaltic concrete modulus of the surface layer derived from deflection basins using the self-iterative program, FPEDD1, represents the modulus value at test temperature. For pavement analysis and overlay design, the asphaltic concrete modulus referring to the design temperature is used. In this study, a default design temperature of 70^{o} F. has been assumed. The correction procedure for temperature sensitivity of the asphaltic concrete modulus is essentially based on the approach used in FHWA-RII overlay design method (Ref 8) for flexible pavements. The following expression is used to obtain the corrected modulus, E1COR, at design temperature.

$$E1COR = E_1 \times CF \tag{E.4}$$

where

E_1 = in situ modulus derived from the self-iterative analysis of deflection basin at the test temperature; and

CF = correction factor.

The correction factor is calculated from the following relationship:

$$CF = E1D/E10 \qquad (E.5)$$

where

E1D = stiffness of the asphalt mix at the design temperature, T_d, an d

E10 = stiffness of the asphalt mix at test temperature, T_t.

E1D and E10 are to be obtained from laboratory M_R tests. It is assumed that the in situ asphalt stiffness has a temperature-M_R relationship parallel to the laboratory-derived curve for the same asphalt mix. In this study a default temperature-M_R curve which is taken from Ref. 8 has been used to calculate the correction factor. No correction is necessary if the test and design temperatures are identical.

Nonlinear behavior of granular layers and subgrade. In situ Young's moduli of elasticity derived from the analysis of a deflection basin are based on the assumption that pavement materials follow the constitutive law of linear elasticity. When dealing with such materials as cement concrete, stabilized materials (using cement, lime, or asphalt) and asphaltic concrete (taking into account temperature sensitivity), linear elasticity is not a bad assumption. Therefore, in situ Young's moduli of these materials determined from deflection basins can be used for pavement analysis and design with layered elastic theory without causing any significant errors in predicting pavement response. However, it has long been recognized that granular layers (base/subbase layers) and subgrades exhibit nonlinear behavior. In RPEDD1 and FPEDD1 an equivalent linear analysis is performed on in situ moduli of materials exhibiting nonlinear behavior. Equivalent linear analysis is based on the concept of strain sensitivity (Ref 5) and drawn from the recent advances in dynamic/seismic response analysis in the geotechnical area.

Dynamic devices such as the Dynaflect or FWD will generate different shear strain amplitudes in pavement layers which can be associated with the determination of nonlinear moduli. Output from ELSYM5 includes maximum shear strain. This strain amplitude is converted to percent strain. The maximum shear strain amplitudes predicted by ELSYM5 for NDT loading and for the design wheel load configuration are compared. The in-situ moduli used in these configurations could be the combination derived from the analysis of a dynamic deflection basin. If the maximum shear strains in the granular layer and subgrade are below the threshold limit, then the corresponding modulus from the analysis of deflection basin is E_{MAX} or strain-independent modulus. This is the case for Dynaflect. Therefore, a nonlinear modulus is to be determined corresponding to the shear strain amplitude determined from the application of a design load. The design load is a simulated, dual wheel, 18-kip, single axle load. The dual wheels are 13.1 inches from center-to-center with a tire pressure of 75 psi. If the dynamic deflection basin data are generated by the Falling Weight Deflectometer, then in the present study the self-iterative procedure for obtaining a nonlinear strain sensitive modulus is omitted because it is assumed that the FWD is capable of generating a peak force on the pavement surface equivalent to the design loads. It has been found that the largest maximum shear strain amplitude at the appropriate depth in every strain sensitive layer caused by both the FWD and design loading configurations are nearly the same (Ref 5). Therefore, in the present study FWD deflection basins corresponding to a peak-force level of around 9000 lbs are used for structural response analyses.

STRUCTURE RESPONSE ANALYSIS

Background

Programs RPEDD1 and FPEDD1 also generate structural response under the design load based on nonlinear in situ moduli (prior applying temperature correction to surface AC modulus in the case of flexible pavements). The programs search for maximum values of the following critical response.

(1) Critical tensile stress/strain at the bottom of the surface layer (tensile strain in the case of flexible pavement and tensile stress for rigid pavements).

(2) Deviator stress on top of subgrade.

(3) Bulk stress at mid-depth of the layer above the subgrade.

(4) Maximum surface deflection under design load (18-kips equivalent single axle is the default value used in this study).

Remaining Life Analysis

The final combination of in situ pavement moduli are assumed to represent effective in situ stiffnesses (Young's moduli) under the design load. In the case of flexible pavements, this step is performed before applying temperature correction. The existing pavement at this test location is again modelled as a layered "linear" elastic system for further evaluation. At this stage of structural evaluation, existing pavement is analyzed for its remaining life at each test location.

The first step is to predict fatigue life of the existing pavement. Fatigue life of a pavement can simply be defined as the maximum number of repetitions of a standard load a pavement can sustain, associated with certain critical response parameter. There is a limiting value associated with the critical response parameter which when exceeded can trigger fatigue cracking. Fatigue cracking initiates at the bottom of either asphaltic concrete layer (in flexible pavements) or surface concrete layer (in rigid pavements) and later appears on the pavement surface. It should be emphasized that a cracked pavement can still carry axle applications without reaching "failure". Here failure is referred to as functional failure of pavement. For this reason, the fatigue equations developed from the analysis of field data generated at AASHO Road Test (Ref 5.6) have been selected for use in this study.

<u>Rigid Pavements</u>. The fatigue equation selected for use in this study is expressed in the following:

$$N_{18} = 46000\ (S/\ \ _{c})^{3.0} \qquad \text{(E.6)}$$ *

where

N_{18} = maximum number of 18 kips equivalent single axle load (ESAL) applications,

S = flexural strength of pavement quality concrete in psi (included in the input data for rigid pavements) and,

$_{c}$ = critical tensile stress at the bottom of concrete layer in psi.

Taute et al. (Ref 4) have developed Equation E.6 from the analysis of AASHO Road Test data (Ref 9) and a study of statewide condition survey data in Texas In the subroutine of remaining life analysis developed in this study, ELSYM5 is used to predict critical tensile stress at the bottom of concrete layer. Recognizing that pavement model based on layered theory does not take into account influence of discontinuities such as cracks, joints, edges, etc., Seeds et al (Ref 10) recommend critical stress factors to adjust the critical tensile stress computed by layered theory before computing N_{18} from Eq. E.6. Therefore, critical

tensile stress, $_{c}$ for use in Equation E.6 is computed by the following *
expression.

$${}_{c} = C_p \times {}_{c}' \qquad (E.7)$$ *

where

$_{c}'$ = critical tensile stress computed by ELSYM5, and *
C_p = critical stress factor. *

Values of C_p recommended by Seeds, et. al. (Ref 10), are selected by the program on the basis of the pavement and shoulder type.

<u>Flexible Pavements</u>. The critical response parameter used in prediction of fatigue life of an existing flexible pavement is critical tensile strain, , at the bottom of asphaltic concrete (AC) layer. FHWA- * ARE Inc's fatigue equation developed from an analysis of data from AASHO Road Test (Ref 9) has been used in FPEDD1.

<u>Remaining Life Estimate</u>. If fatigue life has been computed in terms of allowable number of 18 kips ESAL, N_{18} then an estimate of remaining life of the existing pavement is determined using the following expression.

$$R_L = (1.0 - (n_{18}/N_{18})) \times 100 \qquad (E.8)$$

where

R_L = predicted remaining life of the existing pavement in percent,
N_{18} = predicted fatigue life in 18 kips ESAL, and
n_{18} = past cumulative 18 kips ESAL (entered in the input data).

In equation E.8, n_{18}/N_{18} representing a theoretical damage to the existing pavement is an indication of pavement deterioration due to past repetitions of traffic, n_{18}. Equation E.8 is based on the validity of applying Miner's linear damage hypothesis to estimate fatigue damage in pavements. For the purpose of this study, a very detailed and refined type of remaining life analysis was not done. Therefore the final output generated using special provisions indicates that remaining life analysis at that test location was either not possible or omitted due to one or a combination of the following reasons.

(1) Accumulated past traffic data, n_{18}, in 18 kips ESAL was not entered in the input.

(2) Fatigue life, N_{18} either could not be predicted or set to zero. Remaining life analysis is skipped in this case (e.g., if compressive horizontal strain is computed at the bottom of the asphaltic concrete surface layer.

APPENDIX - F

RESULTS OF NONDESTRUCTIVE DEFLECTION TESTING AND ANALYSIS

Table F-1. In situ Young's moduli determined from deflection basins measured on Site #1. (IH-40, WB)

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI (PSI)			
		P.C.C.	Base	Subbase	Subgrade
1	132	4000000.	247156.	54287.	21398.
2	130	4000000.	293955.	49697.	17098.
3	12901	4000000.	292423.	36561.	12567.
4	128	4000000.	311587.	54357.	22221.
5	127	4000000.	275091.	54455.	26852.
*** MEAN :		4000000.	284043.	49871.	20027.
STD DEV :		0	24330.6	7710.8	5420.2
C V (%) :		0	8.6	15.5	27.1

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI (PSI)			
		P.C.C.	Base	Subbase	Subgrade
1	127	4523000.	549100.	300000.	22610.
2	128	4541000.	850000.	300000.	21360.
3	129	4843000.	850000.	300000.	14770.
4	130	4931000.	850000.	300000.	14860.
5	132	3000000.	100000.	300000.	27850.
*** MEAN :		4367000.	639820.	300000.	20290.
STD DEV :		785341.6	328695.4	0	5559.6
C V (%) :		18.0	51.4	0	27.4

Table F-2. In situ Young's moduli at Site #5 (US-69, NB)

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI (PSI) P.C.C.	Base	Subbase	Subgrade
1	[illegible]	3000000.	90000.	100400.	23830.
[illegible]	[illegible]	3000000.	111500.	117800.	28080.
[illegible]	[illegible]	3000000.	100000.	84200.	29500.
[illegible]	[illegible]	3115000.	126500.	118600.	28310.
	*** MEAN :	3028750.	107000.	105250.	27430.
	STD DEV :	57500.0	15689.7	16353.9	2479.4
	C V (%) :	1.9	14.7	15.5	9.0

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI (PSI) P.C.C.	Base	Subbase	Subgrade
1	100	3409000.	50000.	100400.	28930.
2	200	3000000.	77300.	69500.	34790.
3	300	3000000.	50000.	50000.	32300.
4	400	3000000.	50000.	118600.	33890.
	*** MEAN :	3102250.	56825.	84625.	32478.
	STD DEV :	204500.0	13650.0	30718.1	2575.3
	C V (%) :	6.6	24.0	36.3	7.9

Table F-3. In situ Young's moduli at Site #5 (US-69, SB)

(a) Dynaflect

STATION		P.C.C.	Base	Subbase	Subgrade
		FINAL VALUES OF YOUNGS MODULI (PSI)			
1	400	3000000.	90000.	99900.	23700.
2	300	3836000.	90000.	113800.	31670.
3	200	3000000.	104400.	86300.	20250.
4	100	3000000.	90000.	50000.	25370.
Mean:		3209000.	93600.	87500.	25247.
Std Dev:		418000.0	7200.0	27405.2	4783.1
CV (%):		13.0	7.7	31.3	18.9

(b) FWD

Station		P.C.C.	Base	Subbase	Subgrade
		FINAL VALUES OF YOUNGS MODULI (PSI)			
1	400	3000000.	100000.	99900.	24970.
2	300	3000000.	30000.	179700.	51270.
3	200	3432000.	30000.	86300.	24850.
4	100	3000000.	48400.	37900.	25720.
Mean:		3108000.	52100.	100950.	31703.
Std Dev:		216000.0	33090.4	58857.4	13050.7
CV (%):		6.9	63.5	58.5	41.2

Table F-4. INSITU Young's moduli at site #2 (US69-SB)

(a) DYNAFLECT

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	1.20	100000.	298700.	100000.	23470.
2	1.40	100000.	298700.	100000.	33780.
3	1.60	100000.	298700.	100000.	23470.
4	1.80	100000.	298700.	100000.	27730.
5	2.00	100000.	298700.	143000.	10520.
6	2.20	150900.	450800.	245300.	10320.
7	2.40	76100.	227200.	164200.	31330.
8	3.60	100000.	298700.	100000.	9040.
9	3.80	100000.	298700.	100000.	20320.
10	4.00	55700.	166200.	100000.	10410.
11	4.20	100000.	298700.	100000.	8840.
12	4.40	553700.	700000.	586700.	18020.
13	4.54	100000.	298700.	100000.	20320.
* MEAN :		133569.	325577.	156862.	19044.
STD DEV :		127905.7	128275.0	135992.5	8729.9
C V(%):		95.8	39.4	86.7	45.8

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	4.54	100000.	298700.	100000.	17760.
2	4.40	147700.	441100.	700000.	13110.
3	4.20	100000.	298700.	100000.	10690.
4	4.00	100000.	298700.	100000.	10610.
5	3.80	100000.	298700.	100000.	20980.
6	3.60	52800.	157600.	44500.	9290.
7	2.40	100000.	298700.	100000.	21120.
8	2.20	114100.	340700.	152600.	10740.
9	2.00	90500.	270400.	61400.	9600.
10	1.80	100000.	298700.	100000.	26110.
11	1.60	55600.	165900.	136500.	38990.
12	1.40	100000.	298700.	100000.	27060.
13	1.20	100000.	298700.	100000.	16780.
* MEAN :		96977.	289638.	146538.	17911.
STD DEV :		23531.4	70309.0	168736.8	8848.7
C V(%):		24.3	24.3	115.1	49.4

* E_1 Corrected for design temperature

Table F-5. In situ Young's moduli at Stie #3. (IH-40, EB)

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	7.78	288272.	700000.	160338.	25595.
2	7.70	314787.	700000.	178236.	29138.
3	7.60	243207.	700000.	67254.	12939.
4	7.00	217311.	649055.	98698.	19045.
5	6.00	291857.	700000.	73111.	11971.
6	5.00	161558.	482534.	55107.	24058.
7	4.00	224418.	670282.	119740.	22148.
8	3.00	236773.	700000.	91460.	19099.
9	2.00	226724.	677171.	85162.	18317.
10	1.00	243500.	700000.	116013.	21656.
11	.70	271675.	700000.	167629.	28294.
12	.60	339920.	700000.	50787.	15173.
13	.50	213940.	638985.	59135.	13614.
14	.35	409892.	700000.	59451.	13130.
15	.30	375017.	700000.	61375.	13296.
16	.20	284430.	700000.	62177.	14406.
17	.10	159564.	476580.	61425.	18104.
18	.00	326660.	700000.	79264.	15602.
* MEAN :		268306.	666367.	95854.	18666.
STD DEV :		67746.8	70531.3	38427.6	5473.8
C V(%):		25.2	10.6	40.1	29.3

* E_1 Corrected for design temperature.

Table F-5. In situ Young's moduli at Site #3 (IH-40, EB) (contd).

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	.00	378400.	850000.	103800.	21480.
2	.10	220000.	547500.	50000.	27190.
3	.20	436700.	850000.	53100.	23690.
4	.30	220000.	547500.	50000.	24190.
5	.35	280500.	698000.	50000.	23240.
6	.50	376700.	850000.	84300.	21260.
7	.60	383300.	850000.	59400.	21710.
8	.70	466900.	850000.	85600.	38040.
9	1.00	484000.	850000.	103500.	30650.
10	2.00	388300.	850000.	53000.	28880.
11	3.00	387000.	850000.	50800.	22270.
12	4.00	234800.	584300.	114900.	32840.
13	5.00	461900.	850000.	58600.	31070.
14	6.00	220000.	657100.	50000.	19980.
15	7.00	363000.	850000.	102400.	25480.
16	7.60	451300.	850000.	66300.	22130.
17	7.70	336800.	850000.	240900.	37180.
18	7.78	487900.	850000.	184700.	31130.
* MEAN	:	365417.	782467.	86739.	26801.
STD DEV	:	94330.1	116777.8	51959.9	5591.4
C V(%)	:	25.8	14.9	59.9	20.9

* E_1 Corrected for design temperature.

Table F-6. In situ Young's moduli at Site #3 (IH-40, WB).

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	.10	358053.	700000.	90690.	18574.
2	.20	292907.	700000.	112942.	18461.
3	.30	350204.	700000.	93877.	16152.
4	.35	421758.	700000.	93833.	16043.
5	.50	335690.	700000.	98905.	15773.
6	.60	368289.	700000.	147134.	17868.
7	.70	293269.	700000.	194401.	29176.
8	.80	345996.	700000.	145346.	21428.
9	.90	340148.	700000.	190358.	21128.
10	1.00	257812.	641656.	129794.	26823.
11	1.10	417188.	700000.	202506.	23744.
12	1.20	304023.	700000.	133551.	19948.
13	1.30	338599.	700000.	107367.	13394.
14	1.40	334844.	700000.	110832.	18473.
15	1.50	368957.	700000.	114116.	14569.
16	2.00	393263.	700000.	123327.	18501.
17	3.00	287123.	700000.	128778.	24368.
18	4.00	250888.	624424.	105657.	25984.
19	5.00	201907.	603048.	94971.	18539.
20	6.00	208562.	622924.	52798.	13956.
21	7.00	310040.	700000.	110601.	21165.
22	7.60	342565.	700000.	85442.	11449.
23	7.70	417992.	700000.	227311.	36293.
24	7.78	305806.	700000.	141504.	26914.
* MEAN :		326912.	687169.	126475.	20363.
STD DEV :		59420.9	29858.2	41563.7	5752.7
C V(%):		18.2	4.3	32.9	28.3

* E_1 corrected for design temperature.

Table F-6. In situ Young's moduli at Site #2 (IH-40, WB) (contd).

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	E_1^*	Base	Subgrade
1	.10	479700.	700000.	125200.	24020.
2	.20	434300.	700000.	147300.	30360.
3	.30	417500.	700000.	120300.	23200.
4	.35	412000.	700000.	77800.	24160.
5	.50	462200.	700000.	165100.	23880.
6	.60	570100.	700000.	443400.	29130.
7	.70	438300.	700000.	254700.	45450.
8	.80	489200.	700000.	144500.	30310.
9	.90	652200.	700000.	676400.	28510.
10	1.00	479700.	700000.	91100.	37550.
11	1.10	534700.	700000.	700000.	31470.
12	1.20	435800.	700000.	125800.	30010.
13	1.30	464400.	700000.	139900.	24740.
14	1.40	443100.	700000.	182000.	27850.
15	1.50	477900.	700000.	279700.	20790.
16	2.00	461900.	700000.	112200.	29750.
17	3.00	474100.	700000.	190900.	33750.
18	4.00	370800.	700000.	102000.	31400.
19	5.00	200000.	597400.	80000.	30000.
20	6.00	100000.	298700.	50000.	24000.
21	7.00	321200.	700000.	77600.	30110.
22	7.60	381900.	700000.	94500.	17540.
23	7.70	425900.	700000.	439900.	45970.
24	7.78	471500.	700000.	248300.	30380.
* MEAN :		433267.	679004.	211233.	29347.
STD DEV :		109860.8	83663.0	179118.3	6672.6
C V(%):		25.4	12.3	84.8	22.7

* E_1 corrected for design temperature.

Table F-7. INSITU Young's moduli at site #4 (US69-SB)

(a) DYNAFLECT

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)				
				Selected Fill		
		E_1	E_j	Layer #1	Layer #2	Subgrade
1	0	103000.	198600.	32300.	27100.	15900.
2	20	90000.	173500.	24200.	20000.	17100.
3	40	178600.	344300.	23100.	20000.	20100.
4	60	323600.	624000.	20000.	20000.	37800.
5	80	111900.	215800.	32600.	37600.	19900.
6	100	107500.	207200.	20000.	20000.	21000.
7	120	90000.	173500.	31700.	20000.	11700.
8	140	198500.	382700.	20000.	20000.	25600.
9	160	90000.	173500.	20000.	20000.	30700.
10	180	90200.	174000.	20000.	20000.	26900.
11	200	90000.	173500.	70000.	20000.	15400.
12	220	155500.	299800.	33200.	20000.	18300.
13	240	119800.	231000.	20000.	20000.	16100.
14	260	90000.	173500.	20000.	20000.	15800.
15	280	108600.	209500.	15000.	10000.	34090.
16	300	90000.	173500.	20000.	20000.	13100.
17	320	116700.	225100.	20000.	20000.	26000.
18	340	149900.	289000.	33100.	20000.	26500.
19	360	158200.	305000.	28500.	20000.	40000.
20	380	161700.	311700.	29000.	20000.	22200.
21	400	91600.	176600.	21500.	25400.	18900.
22	420	131100.	252800.	32800.	20000.	11100.
23	460	138500.	267100.	20400.	20000.	21700.
24	480	317600.	612400.	35700.	30300.	17600.
25	500	101100.	195000.	20000.	22400.	23400.
26	540	90000.	173500.	20000.	20000.	16400.
27	560	97100.	187200.	25800.	21100.	17700.
28	580	219200.	654700.	20000.	20000.	24500.
29	600	90000.	268800.	31700.	20000.	10000.
30	620	135800.	405700.	20000.	20000.	19600.
31	640	90000.	268800.	20000.	20000.	67400.
32	660	128100.	382600.	32500.	20000.	27700.
33	680	140900.	420900.	33200.	38100.	27900.
* MEAN :		133173.	279509.	26251.	21576.	22972.
STD DEV :		59189.1	153361.5	9877.9	5148.0	10764.1
C V(%):		44.4	54.9	37.6	23.8	46.9

* E_1 Corrected for design temperature

Table F-7. (continued)

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI (PSI)				
				Selected Fill		
		E_1*	E_j*	Layer #1	Layer #2	Subgrade
1	.00	293800.	566500.	30000.	10000.	27980.
2	.20	169800.	327300.	25600.	10000.	27590.
3	.40	199600.	384900.	16500.	10000.	32660.
4	.60	246400.	475100.	20300.	10000.	33570.
5	.80	112300.	216500.	17300.	10000.	49170.
6	1.00	210600.	406000.	21900.	10000.	38330.
7	1.20	100000.	192800.	30000.	30000.	20480.
8	1.40	108300.	208800.	10700.	10000.	26400.
9	1.60	90000.	173500.	14200.	10000.	34400.
10	1.80	100000.	192800.	30000.	30000.	32310.
11	2.00	90000.	173500.	70000.	22300.	23560.
12	2.20	378500.	700000.	31100.	10400.	21610.
13	2.40	132700.	255900.	10200.	10000.	21980.
14	2.60	100000.	192800.	10000.	10000.	23100.
15	2.80	149500.	288200.	10000.	10000.	33570.
16	3.00	100000.	192800.	10000.	10000.	18950.
17	3.20	100000.	192800.	30000.	30000.	33740.
18	3.40	208900.	402700.	26300.	10000.	37210.
19	3.60	287400.	554100.	28800.	10000.	50010.
20	3.80	267700.	516100.	30000.	10000.	27990.
21	4.00	126800.	244500.	14600.	10000.	23690.
22	4.20	170700.	329100.	44400.	15400.	24260.
23	4.60	100000.	192800.	11000.	11000.	34270.
24	4.80	259500.	500400.	30000.	10000.	31000.
25	5.00	138200.	266500.	16600.	14000.	27960.
26	5.40	100000.	192800.	30000.	30000.	24[illegible]10.
27	5.60	207700.	400400.	30000.	10000.	28670.
28	5.80	90000.	268800.	13600.	11600.	25500.
29	6.00	121000.	361500.	17900.	17500.	18020.
30	6.20	189500.	566000.	30000.	10000.	38690.
31	6.40	187200.	559100.	18900.	10000.	55300.
32	6.60	154000.	459800.	23500.	13800.	33560.
33	6.80	143900.	429800.	15900.	10000.	44350.
* MEAN :		164667.	344988.	23312.	13515.	31360.
STD DEV :		72851.4	148538.2	12043.9	6761.8	8666.0
C V(%):		44.2	43.1	51.7	50.0	28.3

* E_1 corrected to design temperature.

Table F-8. In situ Young's moduli at Site #6 (I-35)

(a) NB - Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)		
		E_1	Base	Subgrade
1	3.50	800000.	251900.	14190.
2	3.00	800000.	220200.	12710.
3	2.50	560200.	163700.	20510.
4	2.00	800000.	309200.	13050.
5	1.50	787800.	241000.	20170.
6	1.00	613100.	287200.	40540.
7	.44	800000.	164300.	11470.
8	.00	412800.	242200.	21060.
	Mean:	696738.	234963.	19213.
	Std Dev:	149858.6	51968.6	9459.0
	CV (%):	21.5	22.1	49.2

(b) NB - FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)		
		E_1	Base	Subgrade
1	.00	500000.	250000.	36830.
2	.44	500000.	250000.	21520.
3	1.00	500000.	250000.	41000.
4	1.05	396000.	508000.	27760.
5	2.00	533200.	574400.	22250.
6	2.05	300800.	171000.	28700.
7	3.00	500000.	250000.	21910.
8	3.05	432400.	250000.	27110.
9	4.00	500000.	250000.	36790.
10	4.05	551500.	357200.	32280.
* MEAN :		471390.	311060.	29615.
STD DEV :		75007.8	130063.6	6911.7
C V(%):		15.9	41.8	23.3

Table F-8. (continued)

(c) SB - FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)		
		E_1	Base	Subgrade
1	.05	500000.	292900.	40710.
2	1.00	716900.	411000.	36680.
3	1.05	562100.	250000.	19430.
4	2.00	466500.	153200.	18320.
5	2.05	500000.	250000.	31530.
6	3.00	597400.	250000.	29240.
7	3.05	625600.	294700.	35850.
8	4.00	500000.	157800.	45090.
9	4.05	556500.	450700.	27300.
* MEAN :		558333.	278922.	31572.
STD DEV :		78941.1	100238.8	9064.8
C V(%):		14.1	35.9	28.7

Table F-9. In situ Young's moduli at Site #7 (US-75, NB)

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	Base	Subbase	Subgrade
1	.00	900000.	159100.	34700.	14350.
2	.50	855500.	141400.	20800.	10000.
3	1.00	868100.	131600.	23500.	10090.
4	1.50	950000.	173900.	25400.	31240.
5	2.00	799900.	176900.	22500.	10000.
6	2.50	701900.	206500.	47000.	18940.
7	2.70	813900.	150000.	32300.	20360.
* MEAN :		834186.	162771.	29514.	16426.
STD DEV :		69945.3	25300.3	9248.3	7834.7
C V(%):		8.4	15.5	31.3	47.7

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	Base	Subbase	Subgrade
1	.00	420400.	375200.	16800.	10690.
2	.50	710300.	385900.	22600.	9130.
3	1.00	485400.	449400.	23000.	11340.
4	1.50	415500.	364800.	16500.	10470.
5	2.00	580300.	700000.	34500.	11490.
6	2.50	698700.	354100.	47100.	21540.
7	2.70	582500.	603700.	10300.	16680.
* MEAN :		556157.	461871.	24400.	13049.
STD DEV :		121501.9	136196.7	12526.5	4438.6
C V(%):		21.8	29.5	51.3	34.[illegible]

Table F-10. In situ Young's moduli at Site #7 (US-75, SB)

(a) Dynaflect

STATICN		FINAL VALUES CF YOUNGS MODULI(PSI)			
		E_1	Base	Subbase	Subgrade
1	2.70	950000.	196400.	35700.	17590.
2	2.50	950000.	295400.	35600.	14370.
3	2.00	931500.	159700.	26200.	10000.
4	1.50	950000.	175200.	11900.	25200.
5	1.00	904300.	126600.	27200.	10070.
6	.50	950000.	129900.	25900.	10000.
7	.00	924300.	129600.	28700.	10640.
* MEAN :		937157.	173257.	27314.	13981.
STD CEV :		17966.0	60006.6	7976.5	5732.9
C V(X):		1.9	34.6	29.2	41.0

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI(PSI)			
		E_1	Base	Subbase	Subgrade
1	2.70	373800.	700000.	27000.	11050.
2	2.50	635500.	700000.	37900.	12620.
3	2.00	591300.	590100.	32200.	10720.
4	1.50	304300.	465300.	24600.	17440.
5	1.00	484500.	404800.	16200.	11210.
6	.50	366100.	408400.	17500.	12180.
7	.00	378600.	278800.	20800.	13230.
* MEAN :		447729.	506771.	25171.	12636.
STD DEV :		125633.2	160926.6	7890.2	2304.3
C V(X):		28.1	31.8	31.3	18.2

Table F-11. In situ Young's moduli at Site #8 (I-35 SB)

(a) Dynaflect

STATION		FINAL VALUES OF YOUNGS MODULI (PSI)			
		E_1	P.C.C.	Subbase	Subgrade
1	10.00	180000.	1500000.	11100.	31320.
2	8.00	180000.	1500000.	11100.	44310.
3	6.00	180000.	1500000.	13100.	17280.
4	4.00	180000.	1500000.	11100.	31440.
5	2.00	208300.	1500000.	13200.	17880.
6	.19	180000.	1500000.	11100.	30730.
* MEAN :		185660.	1500000.	11783.	28827.
STD DEV :		12656.1	0	1059.1	10095.1
C V(%):		6.8	0	9.0	35.0

(b) FWD

STATION		FINAL VALUES OF YOUNGS MODULI (PSI)				
		E_1	E_1*	P.C.C.	Subbase	Subgrade
1	10.0	285800.	700000.	1500000.	87500.	32120.
2	8.0	180000.	537600.	1500000.	87500.	46550.
3	6.0	180000.	537600.	1500000.	69300.	23110.
4	4.0	135300.	404100.	1500000.	65100.	27510.
5	2.0	276424.	700000.	2445900.	63500.	21500.
* MEAN :		211505.	575860.	1689380.	74800.	30158.
STD DEV :		66193.9	125748.4	423466.6	11768.2	10052.3
C V(%):		31.3	21.8	25.1	15.7	33.3

* E_1 corrected for design temperature

Table F-12. Structural response and remaining life analyses, Site #1 (I-40, WB)

(a) Dynaflect

STATION		MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	132.00	4.4	.873E+02	-.167E+01	-.114E+00	5.6
2	130.00	5.1	.876E+02	-.149E+01	0	4.4
3	129.01	6.4	.930E+02	-.132E+01	0	-0
4	128.00	4.2	.834E+02	-.167E+01	-.223E+00	17.6
5	127.00	3.7	.633E+02	-.185E+01	-.859E+00	18.0
					*** MEAN :	9.1
					STD DEV :	8.2
					C V (%) :	89.9

(b) FWD

STATION		MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	127.	3.6	.629E+02	-.140E+01	0	64.6
2	128.	3.7	.554E+02	-.131E+01	0	75.8
3	129.	4.7	.607E+02	-.109E+01	0	68.2
4	130.	4.7	.614E+02	-.109E+01	0	67.1
5	132.	3.7	.723E+02	-.227E+01	0	46.4
					*** MEAN :	64.4
					STD DEV :	10.9
					C V (%) :	17.0

Table F-13. Structural response and remaining life analyses, site No. 5 (US 69, NB).

(a) Dynaflect

	STATION	MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	100	4.4	.840E+02	-.210E+01	0.	43.2
2	200	3.9	.788E+02	-.221E+01	0.	53.0
3	300	3.8	.819E+02	-.230E+01	0.	47.3
4	400	3.8	.785E+02	-.216E+01	0.	53.7
					*** MEAN :	49.3
					STD DEV :	5.0
					C V (%) :	10.1

(b) FWD

	STATION	MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	100	3.8	.902E+02	-.236E+01	0	29.7
2	200	3.5	.835E+02	-.257E+01	-.128E+01	44.1
3	300	3.8	.895E+02	-.259E+01	-.222E+01	31.2
4	400	3.5	.842E+02	-.270E+01	0	42.8
					*** MEAN :	36.9
					STD DEV :	7.6
					C V (%) :	20.4

Table F-14. Structural response and remaining life analyses
Site #5 (US-69, SB)

(a) Dynaflect

STATION		MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	400	4.4	.841E+02	-.299E+01	0.	43.0
2	300	3.4	.866E+02	-.220E+01	0.	37.6
3	200	4.9	.855E+02	-.191E+01	0.	40.0
4	100	4.4	.862E+02	-.218E+01	-.100E+01	34.3
					*** MEAN :	38.7
					STD DEV :	3.7
					C V (%) :	9.6

(b) FWD

STATION		MAX.DEF. (MILS)	H.STRESS (PSI)	DEV.STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	4.00	4.2	.825E+02	-.212E+01	0	46.2
2	3.00	2.8	.824E+02	-.357E+01	-.175E+01	46.3
3	2.00	4.3	.966E+02	-.229E+01	-.318E+00	13.4
4	1.00	4.5	.941E+02	-.232E+01	-.237E+01	20.2
					*** MEAN :	31.5
					STD DEV :	17.2
					C V (%) :	54.7

Table F-15. Structural response and remaining life analyses, Site #2 (US-69, NB)

(a) Dynaflect

STATION		DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	454000	5.1	0	-.618E+01	-.171E+01	100.0
2	4.40	9.4	.296E-04	-.623E+01	0	100.0
3	4.20	13.3	.778E-04	-.658E+01	0	98.0
4	4.00	8.8	.255E-04	-.634E+01	0	100.0
5	3.80	8.5	.163E-03	-.157E+02	-.234E+02	9.0
6	3.60	16.6	.215E-03	-.143E+02	-.213E+02	0
7	2.40	9.5	.170E-03	-.181E+02	-.298E+02	0
8	2.20	9.3	.978E-05	-.723E+01	-.361E+01	100.0
9	2.00	6.5	.120E-04	-.846E+01	-.766E+01	100.0
10	1.80	9.1	.170E-03	-.164E+02	-.253E+02	0
11	1.60	13.6	.238E-03	-.156E+02	-.236E+02	0
12	1.40	14.0	.248E-03	-.159E+02	-.244E+02	0
13	1.20	17.7	.247E-03	-.133E+02	-.183E+02	0

* MEAN : 46,7

STD DEV : 51.1

C V(%): 109.4

(b) FWD

STATION		DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	454000	8.9	0	-.769E+01	-.118E+02	100.0
2	4.40	9.5	0	-.664E+01	-.771E+01	100.0
3	4.20	10.8	.415E-05	-.823E+01	-.924E+01	100.0
4	4.00	9.3	.526E-06	-.693E+01	-.453E+01	100.0
5	3.80	8.5	.600E-04	-.140E+02	-.227E+02	99.5
6	3.60	19.9	0	-.154E+02	-.337E+02	100.0
7	2.40	6.5	.125E-04	-.170E+02	-.339E+02	100.0
8	2.20	17.4	.863E-05	-.176E+02	-.368E+02	100.0
9	2.00	11.2	.887E-04	-.144E+02	-.233E+02	96.1
10	1.80	7.6	.262E-04	-.163E+02	-.312E+02	100.0
11	1.60	12.2	.784E-04	-.130E+02	-.200E+02	97.9
12	1.40	12.0	.794E-04	-.150E+02	-.256E+02	97.8
13	1.20	21.2	0	-.129E+02	-.311E+02	100.0

* MEAN : 99.3

STD DEV : 1.2

C V(%): 1.3

Table F-16 Structural response and remaining life analyses, Site #2 (US-69, SB)

(a) Dynaflect

STATION		DEF. (MILS)	MAX.H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	120	12.9	.717E-04	-.140E+02	-.234E+02	98.7
2	140	10.7	.813E-04	-.158E+02	-.274E+02	97.5
3	160	12.9	.717E-04	-.140E+02	-.234E+02	98.7
4	180	11.8	.762E-04	-.148E+02	-.253E+02	98.2
5	200	18.8	.349E-05	-.926E+01	-.136E+02	100.0
6	220	15.7	0.	-.734E+01	-.913E+01	100.0
7	240	10.3	.167E-04	-.144E+02	-.277E+02	100.0
8	360	22.4	.399E-04	-.950E+01	-.120E+02	99.9
9	380	14.0	.674E-04	-.132E+02	-.218E+02	99.1
10	400	22.7	.670E-05	-.111E+02	-.199E+02	100.0
11	420	22.7	.391E-04	-.941E+01	-.117E+02	99.9
12	440	7.5	0.	-.574E+01	0.	100.0
13	454	14.0	.674E-04	-.132E+02	-.218E+02	99.1
					* MEAN :	99.3
					STD DEV :	0.8
					C V(%):	0.8

(b) FWD

STATION		DEF. (MILS)	MAX.H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	454.000	15.1	.633E-04	-.126E+02	-.202E+02	99.3
2	4.40	10.5	0	-.582E+01	-.152E+02	100.0
3	4.20	20.3	.461E-04	-.102E+02	-.140E+02	99.9
4	4.00	20.4	.458E-04	-.102E+02	-.139E+02	99.9
5	3.80	13.8	.684E-04	-.134E+02	-.221E+02	99.0
6	3.60	30.1	.170E-03	-.130E+02	-.206E+02	0
7	2.40	13.7	.686E-04	-.134E+02	-.222E+02	99.0
8	2.20	17.5	0	-.882E+01	-.124E+02	100.0
9	2.00	24.9	.126E-03	-.112E+02	-.149E+02	76.4
10	1.80	12.2	.746E-04	-.145E+02	-.246E+02	98.4
11	1.60	10.5	.294E-04	-.166E+02	-.323E+02	100.0
12	1.40	12.0	.756E-04	-.147E+02	-.250E+02	98.3
13	1.20	15.5	.615E-04	-.123E+02	-.195E+02	99.4
					* MEAN :	90.0
					STD DEV :	27.8
					C V(%):	30.9

Table F-17. Structural response and remaining life analyses
Site #3 (I-40, EB)

(a) Dynaflect

STATION	DEF.	MAX. H. STRAIN		DEV. STRESS	B. STRESS	REM. LIFE
	(MILS)		IN./IN.	(PSI)	(PSI)	PERCENT
1	7.78	8.5	.779E-04	-.846E+01	-.323E+01	98.3
2	7.70	7.7	.700E-04	-.853E+01	-.346E+01	99.0
3	7.60	16.3	.182E-03	-.786E+01	-.155E+01	0
4	7.00	12.1	.128E-03	-.875E+01	-.421E+01	77.5
5	6.00	16.3	.166E-03	-.716E+01	0	16.2
6	5.00	11.2	.134E-03	-.103E+02	-.930E+01	72.4
7	4.00	10.5	.105E-03	-.889E+01	-.467E+01	91.9
8	3.00	12.1	.138E-03	-.876E+01	-.431E+01	67.4
9	2.00	12.7	.148E-03	-.883E+01	-.454E+01	53.4
10	1.00	10.5	.109E-03	-.871E+01	-.407E+01	90.3
11	.70	8.1	.738E-04	-.881E+01	-.448E+01	98.7
12	.60	13.1	.135E-03	-.733E+01	0	71.3
13	.50	16.6	.205E-03	-.855E+01	-.390E+01	0
14	.35	15.5	.175E-03	-.735E+01	-.104E+01	0
15	.30	15.4	.176E-03	-.747E+01	-.114E+01	0
16	.20	15.2	.186E-03	-.815E+01	-.300E+01	0
17	.10	14.7	.208E-03	-.102E+02	-.883E+01	0
18	0	13.4	.151E-03	-.775E+01	-.129E+01	48.9
					* MEAN :	49.2
					STD DEV :	41.0
					C V(%):	83.5

Table F-17. Structural response and remaining life analyses (contd)
Site #3 (I-40, EB)

(b) FWD

STATION		DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	0	10.1	.117E-03	-.813E+01	-.297E+01	86.0
2	.10	11.6	.223E-03	-.118E+02	-.137E+02	0
3	.20	10.9	.173E-03	-.972E+01	-.911E+01	0
4	.30	12.4	.225E-03	-.113E+02	-.123E+02	0
5	.35	12.1	.210E-03	-.106E+02	-.109E+02	0
6	.50	10.7	.137E-03	-.854E+01	-.419E+01	68.1
7	.60	11.4	.172E-03	-.956E+01	-.747E+01	0
8	.70	7.3	.124E-03	-.105E+02	-.104E+02	81.4
9	1.00	7.6	.110E-03	-.999E+01	-.606E+01	90.0
10	2.00	9.9	.175E-03	-.108E+02	-.119E+02	0
11	3.00	11.7	.156E-03	-.980E+01	-.913E+01	0
12	4.00	8.4	.111E-03	-.105E+02	-.968E+01	89.3
13	5.00	9.0	.157E-03	-.105E+02	-.114E+02	37.0
14	6.00	13.8	.227E-03	-.104E+02	-.999E+01	0
15	7.00	9.2	.118E-03	-.889E+01	-.500E+01	85.2
16	7.60	10.7	.152E-03	-.980E+01	-.629E+01	46.3
17	7.70	6.1	.498E-04	-.675E+01	-.449E+01	99.8
18	7.78	6.8	.684E-04	-.722E+01	-.137E+01	99.1

* MEAN :	43.5
STD DEV :	42.9
C V(%):	98.7

Table F-18. Structural response and remaining life analyses, Site #3 (I-40, WB)

(a) Dynaflect

STATION		DEF.MAX. (MILS)	H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE FERCENT
1	.10	11.5	.132E-03	-.797E+01	-.199E+01	73.7
2	.20	11.3	.112E-03	-.784E+01	-.106E+01	88.9
3	.30	12.5	.130E-03	-.743E+01	0	75.9
4	.35	12.2	.126E-03	-.713E+01	0	79.6
5	.50	12.6	.125E-03	-.732E+01	0	80.1
6	.60	10.5	.861E-04	-.688E+01	0	97.1
7	.70	7.6	.625E-04	-.850E+01	-.348E+01	99.5
8	.80	9.5	.871E-04	-.762E+01	-.189E+00	97.0
9	.90	9.1	.645E-04	-.712E+01	0	99.4
10	1.00	9.0	.977E-04	-.921E+01	-.574E+01	94.5
11	1.10	8.0	.619E-04	-.707E+01	0	99.5
12	1.20	10.3	.947E-04	-.774E+01	-.631E+00	95.3
13	1.30	13.7	.117E-03	-.661E+01	0	86.0
14	1.40	11.1	.113E-03	-.766E+01	-.508E+00	88.4
15	1.50	12.6	.110E-03	-.665E+01	0	89.9
16	2.00	10.6	.101E-03	-.721E+01	0	93.4
17	3.00	9.3	.964E-04	-.865E+01	-.388E+01	94.3
18	4.00	9.6	.120E-03	-.957E+01	-.691E+01	84.2
19	5.00	12.5	.133E-03	-.886E+01	-.457E+01	72.5
20	6.00	17.0	.227E-03	-.896E+01	-.542E+01	0
21	7.00	10.4	.114E-03	-.829E+01	-.273E+01	88.0
22	7.60	15.8	.143E-03	-.650E+01	0	60.7
23	7.70	6.0	.551E-04	-.836E+01	-.288E+01	99.7
24	7.78	8.5	.896E-04	-.873E+01	-.413E+01	96.5
					* MEAN :	84.8
					STD DEV :	20.8
					C V(%):	24.5

Table F-18. Structural response and remaining life analyses (contd) Site #3 (I-40, WB).

(b) FWD

STATION		DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	.10	8.7	.967E-04	-.778E+01	-.126E+01	94.8
2	.20	7.4	.848E-04	-.848E+01	-.344E+01	97.4
3	.30	9.2	.102E-03	-.796E+01	-.177E+01	93.1
4	.35	9.9	.141E-03	-.906E+01	-.618E+01	53.9
5	.50	8.2	.765E-04	-.729E+01	0	98.4
6	.60	5.7	.240E-04	-.598E+01	0	100.0
7	.70	5.1	.491E-04	-.891E+01	-.478E+01	99.8
8	.80	7.3	.851E-04	-.831E+01	-.299E+01	97.3
9	.90	5.2	.118E-04	-.515E+01	0	100.0
10	1.00	7.2	.118E-03	-.102E+02	-.964E+01	85.3
11	1.10	5.0	.877E-05	-.566E+01	0	100.0
12	1.20	7.7	.963E-04	-.875E+01	-.450E+01	94.9
13	1.30	8.4	.885E-04	-.772E+01	-.827E+00	96.7
14	1.40	7.4	.696E-04	-.772E+01	-.552E+00	99.0
15	1.50	8.0	.426E-04	-.592E+01	0	99.9
16	2.00	7.9	.105E-03	-.887E+01	-.517E+01	92.1
17	3.00	6.4	.663E-04	-.821E+01	-.235E+01	99.3
18	4.00	8.1	.117E-03	-.970E+01	-.760E+01	85.8
19	5.00	5.5	.158E-03	-.113E+02	-.119E+02	34.7
20	6.00	7.1	.259E-03	-.128E+02	-.159E+02	0.0
21	7.00	9.1	.149E-03	-.104E+02	-.989E+01	52.3
22	7.60	11.7	.127E-03	-.758E+01	-.602E+00	78.6
23	7.70	4.6	.226E-04	-.800E+01	-.255E+01	100.0
24	7.78	6.5	.501E-04	-.735E+01	0	99.8

* MEAN :	86.0
STD DEV :	24.8
C V(%):	28.9

Table F-19. Structural response and remaining life analyses, Site #4 (US-69, SB),

(a) Dynaflect

STATION		DEFL.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	H.STRESS (PSI)	REM. LIFE PERCENT
1	0	13.8	.184E-03	-.229E+01	-.811E+01	25.0
2	.20	15.4	.231E-03	-.261E+01	-.928E+01	0
3	.40	11.3	.166E-03	-.249E+01	-.712E+01	56.0
4	.60	7.4	.117E-03	-.285E+01	-.729E+01	92.9
5	.80	12.0	.175E-03	-.246E+01	-.986E+01	42.4
6	1.00	14.0	.231E-03	-.286E+01	-.894E+01	0
7	1.20	16.7	.198E-03	-.208E+01	-.640E+01	0
8	1.40	10.3	.162E-03	-.275E+01	-.779E+01	60.7
9	1.60	13.7	.252E-03	-.336E+01	-.999E+01	0
10	1.80	14.1	.252E-03	-.321E+01	-.978E+01	0
11	2.00	13.2	.109E-03	-.205E+01	-.339E+01	94.9
12	2.20	11.5	.154E-03	-.227E+01	-.574E+01	70.1
13	2.40	14.6	.220E-03	-.251E+01	-.819E+01	0
14	2.60	16.4	.254E-03	-.263E+01	-.890E+01	0
15	2.80	14.1	.265E-03	-.358E+01	-.778E+01	0
16	3.00	17.4	.254E-03	-.242E+01	-.855E+01	0
17	3.20	12.7	.221E-03	-.305E+01	-.910E+01	0
18	3.40	10.3	.156E-03	-.271E+01	-.658E+01	67.8
19	3.60	9.2	.162E-03	-.322E+01	-.782E+01	61.5
20	3.80	10.9	.160E-03	-.254E+01	-.665E+01	63.4
21	4.00	14.8	.241E-03	-.276E+01	-.997E+01	0
22	4.20	14.9	.168E-03	-.187E+01	-.520E+01	52.8
23	4.60	12.4	.200E-03	-.276E+01	-.830E+01	0
24	4.80	8.8	.101E-03	-.186E+01	-.561E+01	96.6
25	5.00	13.7	.237E-03	-.299E+01	-.976E+01	0
26	5.40	16.2	.254E-03	-.267E+01	-.896E+01	0
27	5.60	14.5	.215E-03	-.257E+01	-.815E+01	0
28	5.80	10.0	.153E-03	-.265E+01	-.748E+01	71.2
29	6.00	17.6	.198E-03	-.193E+01	-.610E+01	0
30	6.20	13.0	.204E-03	-.267E+01	-.824E+01	0
31	6.40	11.9	.252E-03	-.409E+01	-.109E+02	0
32	6.60	10.9	.169E-03	-.285E+01	-.707E+01	51.4
33	6.80	9.6	.156E-03	-.271E+01	-.978E+01	67.9

* MEAN : 30.6

STD DEV : 34.7

C V(%): 113.5

Table F-19. Structural response and remaining life analyses (contd).

Site #4 (US-69, SB)

(b) FWD

STATION		DEF. MAX. (MILS)	H. STRAIN IN./IN.	DEV. STRESS (PSI)	B. STRESS (PSI)	REM. LIFE PERCENT
1	0	8.7	.117E-03	-.257E+01	-.322E+01	92.6
2	.20	11.1	.169E-03	-.294E+01	-.455E+01	51.1
3	.40	10.7	.177E-03	-.313E+01	-.629E+01	39.2
4	.60	9.4	.146E-03	-.298E+01	-.529E+01	77.5
5	.80	12.6	.246E-03	-.387E+01	-.766E+01	0
6	1.00	9.5	.157E-03	-.320E+01	-.543E+01	67.2
7	1.20	12.9	.194E-03	-.263E+01	-.944E+01	2.7
8	1.40	16.0	.297E-03	-.340E+01	-.852E+01	0
9	1.60	15.6	.300E-03	-.379E+01	-.834E+01	0
10	1.80	11.4	.194E-03	-.311E+01	-.102E+02	2.6
11	2.00	11.5	.111E-03	-.248E+01	-.472E+01	94.6
12	2.20	7.5	.988E-04	-.253E+01	-.316E+01	97.0
13	2.40	15.4	.265E-03	-.311E+01	-.792E+01	0
14	2.60	17.3	.320E-03	-.334E+01	-.870E+01	0
15	2.80	13.2	.243E-03	-.341E+01	-.825E+01	0
16	3.00	16.2	.321E-03	-.315E+01	-.842E+01	0
17	3.20	11.3	.194E-03	-.315E+01	-.102E+02	2.6
18	3.40	9.3	.149E-03	-.313E+01	-.466E+01	75.2
19	3.60	7.4	.119E-03	-.320E+01	-.435E+01	92.3
20	3.80	9.0	.124E-03	-.263E+01	-.332E+01	90.2
21	4.00	14.4	.245E-03	-.314E+01	-.708E+01	0
22	4.20	9.9	.129E-03	-.248E+01	-.352E+01	88.0
23	4.60	15.4	.307E-03	-.366E+01	-.930E+01	0
24	4.80	9.8	.126E-03	-.276E+01	-.353E+01	89.4
25	5.00	12.5	.219E-03	-.316E+01	-.802E+01	0
26	5.40	12.2	.194E-03	-.281E+01	-.976E+01	2.7
27	5.60	9.8	.143E-03	-.281E+01	-.364E+01	79.7
28	5.80	16.4	.304E-03	-.340E+01	-.871E+01	0
29	6.00	14.4	.229E-03	-.270E+01	-.819E+01	0
30	6.20	9.4	.149E-03	-.320E+01	-.426E+01	74.7
31	6.40	9.5	.176E-03	-.364E+01	-.662E+01	41.2
32	6.60	10.7	.1[illegible]1E-03	-.316E+01	-.674E+01	31.2
33	6.80	11.6	.219E-03	-.364E+01	-.739E+01	0

* MEAN :	36.1
STD DEV :	40.0
C V(%):	110.8

Table F-20. Structural response and remaining life analyses at Site #6 (I-35)

(a) NB - Dynaflect

STATION	DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	3.50	6.7	.318E-04	-.181E+01	0.	100.0
2	3.00	7.4	.357E-04	-.177E+01	0.	100.0
3	2.50	6.3	.480E-04	-.267E+01	0.	99.8
4	2.00	6.8	.263E-04	-.164E+01	0.	100.0
5	1.50	5.5	.330E-04	-.219E+01	0.	100.0
6	1.00	3.7	.291E-04	-.317E+01	-.101E+01	100.0
7	.44	8.4	.450E-04	-.183E+01	0.	99.9
8	0	6.1	.340E-04	-.262E+01	0.	100.0

Mean: 99.9

Std Dev: .1

CV (%): .1

(b) NB - FWD

STATION		DEF.MAX.H. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	0	4.3	.336E-04	-.329E+01	-.143E+01	100.0
2	.44	5.7	.331E-04	-.250E+01	0.	100.0
3	1.00	4.0	.336E-04	-.347E+01	-.199E+01	100.0
4	1.05	4.6	.124E-04	-.251E+01	-.336E+00	100.0
5	2.00	4.7	.114E-04	-.202E+01	0.	100.0
6	2.05	6.0	.494E-04	-.363E+01	-.251E+01	99.8
7	3.00	5.7	.332E-04	-.253E+01	0.	100.0
8	3.05	5.2	.333E-04	-.292E+01	-.319E+00	100.0
9	4.00	4.3	.336E-04	-.329E+01	-.142E+01	100.0
10	4.05	4.1	.229E-04	-.274E+01	0.	100.0

* MEAN : 100.0

STD DEV : .1

C V(%): .1

Table F-21. Structural response and remaining life analyses at Site #6 (I-35) (contd).

(c) SB - FWD

STATION		DEF.MAX.D. (MILS)	STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	005	3.9	.287E-04	-.332E+01	-.163E+01	100.0
2	100	3.5	.201E-04	-.264E+01	0.	100.0
3	105	5.9	.330E-04	-.231E+01	0.	100.0
4	200	7.1	.524E-04	-.268E+01	0.	99.7
5	205	4.6	.334E-04	-.304E+01	-.623E+00	100.0
6	300	4.6	.331E-04	-.280E+01	0.	100.0
7	305	3.9	.283E-04	-.294E+01	-.258E+00	100.0
8	400	4.3	.501E-04	-.411E+01	-.368E+01	99.8
9	405	4.3	.170E-04	-.236E+01	0.	100.0

* MEAN :	99.9
STD DEV :	.1
C V(%):	.1

Table F-22. Structural response and remaining life analyses.
Site #7 (US-75, NB)

(a) Dynaflect

STATION		DEF.MAX. (MILS)	H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	0	9.2	.663E-04	-.184E+01	-.298E+01	99.8
2	50	12.6	.744E-04	-.179E+01	-.289E+01	99.7
3	100	12.3	.815E-04	-.176E+01	-.278E+01	99.5
4	150	7.7	.609E-04	-.263E+01	-.518E+01	99.9
5	200	11.6	.530E-04	-.172E+01	-.269E+01	99.9
6	250	7.2	.433E-04	-.192E+01	-.326E+01	100.0
7	270	8.6	.712E-04	-.218E+01	-.403E+01	99.8
					* MEAN :	99.8
					STD DEV :	.2
					C V(X):	.2

(b) FWD

STATION		DEF.MAX. (MILS)	H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	0	10.8	0	-.185E+01	-.324E+01	100.0
2	50	10.2	.457E-05	-.156E+01	-.229E+01	100.0
3	100	9.3	0	-.174E+01	-.287E+01	100.0
4	150	11.1	0	-.185E+01	-.324E+01	100.0
5	200	7.5	0	-.151E+01	-.212E+01	100.0
6	250	6.0	.147E-04	-.193E+01	-.331E+01	100.0
7	270	8.5	0	-.203E+01	-.386E+01	100.0
					* MEAN :	100.0
					STD DEV :	.0
					C V(X):	.0

Table F-23. Structural response and remaining life analyses, Site #7 (US-75, SB).

(a) Dynaflect

STATION		DEF. (MILS)	MAX.H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	270	6.8	.528E-04	-.213E+01	-.350E+01	99.9
2	25	6.4	.263E-04	-.194E+01	-.289E+01	100.0
3	200	11.3	.649E-04	-.167E+01	-.247E+01	99.9
4	150	9.2	.596E-04	-.269E+01	-.525E+01	99.9
5	100	11.9	.855E-04	-.171E+01	-.258E+01	99.4
6	50	12.0	.836E-04	-.171E+01	-.259E+01	99.5
7	0	11.4	.837E-04	-.172E+01	-.261E+01	99.5
					* MEAN :	99.7
					STD DEV :	.3
					C V(%):	.3

(b) FWD

STATION		DEF. (MILS)	MAX.H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)	REM. LIFE PERCENT
1	270	6.6	0	-.186E+01	-.279E+01	100.0
2	250	5.4	0	-.179E+01	-.247E+01	100.0
3	200	8.1	0	-.152E+01	-.212E+01	100.0
4	150	6.7	0	-.229E+01	-.411E+01	100.0
5	100	10.4	0	-.185E+01	-.326E+01	100.0
6	50	10.1	0	-.194E+01	-.350E+01	100.0
7	0	10.2	.357E-05	-.204E+01	-.369E+01	100.0
					* MEAN :	100.0
					STD DEV :	.0
					C V(%):	.0

Table F-24. Structural response analyses (Dynaflect)

(a) Site #8, (I-35, NB)

STATION		DEF.MAX. (MILS)	H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)
1	0	5.9	0	-.388E+01	-.648E+01
2	100	5.8	0	-.394E+01	-.657E+01
3	200	6.5	0	-.258E+01	-.422E+01
4	300	6.5	0	-.274E+01	-.437E+01
5	400	5.7	0	-.399E+01	-.668E+01
6	500	7.4	0	-.309E+01	-.499E+01
7	600	8.0	0	-.281E+01	-.431E+01
8	700	6.2	0	-.368E+01	-.611E+01
9	800	5.1	0	-.451E+01	-.756E+01
10	900	6.3	0	-.364E+01	-.604E+01
11	1000	4.9	0	-.467E+01	-.783E+01
12	1100	7.0	0	-.271E+01	-.423E+01
13	1190	7.2	0	-.303E+01	-.489E+01

(b) Site #8, (I-35, SB)

STATION		DEF.MAX. (MILS)	H. STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)
1	10.00	5.6	0	-.410E+01	-.687E+01
2	8.00	4.7	0	-.484E+01	-.812E+01
3	6.00	7.7	0	-.292E+01	-.455E+01
4	4.00	5.6	0	-.411E+01	-.689E+01
5	2.00	7.3	0	-.293E+01	-.459E+01
6	.20	0	0	0	0
7	.19	5.6	0	-.407E+01	-.680E+01

Table F-25. Structural response analyses (FWD)

(a) Site #8 (I-35, NB)

STATION	DEF.MAX.H. (MILS)		STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)
1	110	4.4	0	-.214E+01	-.335E+01
2	100	3.8	0	-.463E+01	-.738E+01
3	90	5.6	0	-.269E+01	-.359E+00
4	80	4.4	0	-.499E+01	-.839E+01
5	70	5.5	0	-.314E+01	-.174E+00
6	60	6.6	0	-.276E+01	-.312E+00
7	50	6.2	0	-.288E+01	-.266E+00
8	40	5.6	0	-.326E+01	-.199E+01
9	30	5.6	0	-.262E+01	-.477E+00
10	20	5.0	0	-.203E+01	-.169E+01
11	10	4.5	0	-.266E+01	-.181E+01
12	0	5.3	0	-.299E+01	-.258E+01

(b) Site #8 (I-35, SB)

STATION	DEF.MAX.H. (MILS)		STRAIN IN./IN.	DEV.STRESS (PSI)	B.STRESS (PSI)
1	100	4.5	0	-.321E+01	-.677E+00
2	80	4.2	0	-.414E+01	-.338E+01
3	60	6.1	0	-.291E+01	-.255E+00
4	40	6.0	0	-.331E+01	-.158E+01
5	20	5.4	0	-.231E+01	-.570E+00