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16. Abstract

This report summarizes an investigation of premature distress and failure of an asphalt concrete overlay on IH-10 near Columbus, Texas. The primary objectives of the study were to determine the probable causes of the distress and to make recommendations which could alleviate future problems. The investigation involved an analysis of construction records and laboratory test results performed during and after construction. In addition, specimens and material were obtained from the roadway for use in a laboratory evaluation. The sampling program included collection of cores, slabs, and stockpile or pit materials. The report contains a description and summary of the pavement and distress, construction procedures, and mixture characteristics, along with the findings related to the probable causes of the distress and recommendations.

The basic causes were that (1) all aggregates and the resulting aggregate-asphalt combinations were highly susceptible to moisture damage, (2) the anti-stripping agent was not effective, and (3) the project may have had localized sections which were over-asphalted.

All findings indicated that construction was in compliance with specifications. Nevertheless, a series of recommendations, contained in the report, was felt to be justified.

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INVESTIGATION OF PREMATURE DISTRESS IN CONVENTIONAL ASPHALT MATERIALS ON INTERSTATE 10 AT COLUMBUS, TEXAS

by

Thomas W. Kennedy
Robert B. McGennis
Freddy L. Roberts

Research Report Number 313-1

Investigate Distress of Recycled and Conventional Asphalt
Base Material and Surfaces on Interstate 10, Texas

Research Project 3-9-81-313

conducted for

Texas
State Department of Highways and Public Transportation

by the

CENTER FOR TRANSPORTATION RESEARCH
BUREAU OF ENGINEERING RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

August 1982

PREFACE

This is the first in a series of reports that deal with a study of premature failures of recycled and conventional asphalt concrete mixtures on IH-10 in Texas, the primary objectives of which were to determine the most probable causes of the distress and to make recommendations which could alleviate future problems. This first report is concerned with the failure of a conventional overlay mixture at Columbus in the Yoakum District, District 13. The report summarizes the pavement history, construction procedures, mixture characteristics, evaluation and laboratory study procedures, findings related to the probable causes of the distress, and recommendations.

This report was completed with the assistance of many people. Special appreciation is extended to Messrs. James N. Anagnos, Pat Hardeman, and Eugene Betts for their assistance in conducting the field and laboratory studies and to Mr. Bruce Bayless, Resident Engineer, District 13, and Mr. Charles W. Chaffin, contact man with the Texas State Department of Highways and Public Transportation. In addition, special appreciation is extended to the other members of the SDHPT Advisory Panel, including Messrs. Billy R. Neeley, Walter W. Chambers, Robert L. Mikulin, Kenneth D. Hankins, Warren N. Dudley, and Clinton B. Bond. Appreciation is also extended to the staff of the Center for Transportation Research who assisted with preparation of the manuscript.

Thomas W. Kennedy
Robert B. McGennis
Freddy L. Roberts

LIST OF REPORTS

Report No. 313-1, "Investigation of Premature Distress on Conventional Asphalt Materials on Interstate 10 at Columbus, Texas," by Thomas W. Kennedy, Robert B. McGennis, and Freddy L. Roberts, summarizes a study of construction records and laboratory tests on both field specimens and laboratory prepared specimens to determine the causes of premature and severe failure of an asphalt concrete overlay on IH-10 near Columbus, Texas.

ABSTRACT

This report summarizes an investigation of premature distress and failure of an asphalt concrete overlay on IH-10 near Columbus, Texas. The primary objectives of the study were to determine the probable causes of the distress and to make recommendations which could alleviate future problems. The investigation involved an analysis of construction records and laboratory test results performed during and after construction. In addition, specimens and material were obtained from the roadway for use in a laboratory evaluation. The sampling program included collection of cores, slabs, and stockpile or pit materials. The report contains a description and summary of the pavement and distress, construction procedures, and mixture characteristics, along with the findings related to the probable causes of the distress and recommendations.

The basic causes were that (1) all aggregates and the resulting aggregate-asphalt combinations were highly susceptible to moisture damage, (2) the antistripping agent was not effective, and (3) the project may have had localized sections which were over-asphalted.

All findings indicated that construction was in compliance with specifications. Nevertheless, a series of recommendations, contained in the report, was felt to be justified.

KEY WORDS: premature distress, construction records, compaction, rutting, shoving, stripping, gradations, condition surveys, stability, tensile strength, freeze-thaw pedestal test, compaction temperature.

SUMMARY

This report summarizes an investigation of premature distress and failure of an asphalt concrete overlay on IH-10 near Columbus, Texas. The primary objectives of the study were to determine the probable causes of the distress and to make recommendations which could alleviate future problems. The investigation involved an analysis of construction records and laboratory test results performed during and after construction.

The distress was in the form of rutting and shoving and was directly related to the Type B base course material which was placed using three mixture designs (designs 1, 2, and 5). Major distress began in the design 1 material and was concentrated in designs 1 and 2. Essentially no distress occurred in design 5 although a small amount of rutting had occurred at the time of the last condition survey. Designs 1 and 2 were similar with design 2 having only 0.2 percent more asphalt, while design 5 was markedly different, containing a different field sand and asphalt content.

The basic causes of the distress were:

(1) All aggregates and the resulting aggregate-asphalt combinations were highly susceptible to moisture damage.

(2) The antistripping agent was not effective in preventing moisture damage.

(3) In localized areas, the mixtures may have had higher asphalt contents, which could have contributed to the failures.

Other findings related to the distress and performance of the pavement and the various designs are as follows:

(1) Base course mixtures placed after the winter, design 5 and a portion of design 2, had a lower field moisture content than the rest of the project. This was due to the fact that these mixtures had higher densities, were not exposed during the winter, and were covered soon after being placed in the spring.

(2) The sand source used for designs 1 and 2 contained localized areas of dirty sand, although there is no evidence to indicate that this was a definite and direct cause of the severe distress.

(3) The asphalt contents for the entire project were essentially within the specified tolerance of ±0.5 percent. Mixture designs 1 and 2 tended to have slightly higher relative asphalt contents with larger percentages of the asphalt contents above the design value, while design 5 was substantially below the design value.

(4) The tensile strengths and the portion of the tensile strength retained after being subjected to moisture were much greater for design 5. This is attributed to the increased density which produced higher strengths and reduced moisture penetration.

(5) The Hveem stabilities on the cores for design 5 were higher than for designs 1 and 2, although there were essentially no differences between Hveem stabilities of laboratory prepared, job control specimens.

(6) Laboratory studies and field observations indicated that all aggregates were highly susceptible to moisture damage and that the antistripping agent was not effective in preventing moisture damage.

All information and findings indicate that construction was in compliance with specifications and the quality of construction was satisfactory. Nevertheless, a series of recommendations related to testing, design, and construction is justified. These recommendations are summarized in the report.

IMPLEMENTATION STATEMENT

Based on findings of the study the following recommendations have been developed to ensure adequate performance of asphalt mixtures and asphalt concrete overlay mixtures for future construction.

(1) All asphalt-aggregate combinations including mixtures containing proposed antistripping agents should be tested to determine their susceptibility to stripping and moisture damage. Suggested test methods are Texas Freeze-Thaw Pedestal Test, Boiling Test, and Wet-Dry Indirect Tensile Tests.

(2) Specified or required Hveem stability values should be increased to 35 or 40 for high volume highways.

(3) Specifications requiring a minimum level of compaction should be developed and definite density control requirements need to be implemented.

(4) Test procedures as used by D-9, the districts, and the residencies should be compared and checked periodically to ensure that satisfactory and comparable results are being obtained.

(5) The specified tolerance of ±0.5 percent for asphalt content should be evaluated to determine whether tighter control is needed and is possible.

(6) The effect of aggregate gradation and the specified gradations should be evaluated to determine whether a tighter specification is required. Attention should be paid to the amount of material in the interval between sieves no. 40 and 80, the amount of fines, and the general shape of the gradation curve.

(7) Consideration should be given to establishing a construction sequence which minimizes exposure.

(8) Although not an apparent factor in this study, the viscosity of the asphalt should be evaluated to determine the long-term effects on stability and strength. Special emphasis needs to be placed on the influence of plant temperatures, especially with respect to drum dryers which are more variable and operate at lower temperatures.

(9) Design and job control testing needs to be increased to ensure adequate design and construction. In addition, consideration should be given to implementing and, if necessary, developing, new tests which could provide more meaningful information with respect to material properties and pavement performance.

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APPENDICES

CHAPTER 1. PROJECT AND EVALUATION

INTRODUCTION

In the fall of 1979, an overlay rehabilitation project was undertaken on IH-10 in District 13 near Columbus, Texas, Colorado County. The project was nine miles long and extended from approximately six miles west to three miles east of Columbus. It consisted of overlaying a continuously reinforced concrete pavement (CRCP) with hot mix asphalt concrete.

In June 1980, prior to completion of the contract, distress began to develop on certain sections of the highway (Figs 1 through 4). Distress was in the form of rutting, shoving, and bleeding. Initially, distress occurred in small areas of the outside westbound lanes; however, distress subsequently developed in other areas. Minor maintenance operations were performed on the most highly distressed areas, and signs were installed to advise traffic to change lanes to avoid these distressed areas.

The purpose of the study summarized in this report was to determine the probable causes of the distress and to make recommendations which will alleviate future problems. The distress is described and the pertinent history, design and construction information, and the test program is summarized in the remainder of Chapter 1. Chapter 2 contains an evaluation of the project and the data and information secured, and Chapter 3 contains the findings, conclusions, and recommendations.

PROJECT DESCRIPTION

Figure 5 summarizes the existing and proposed pavement structures. In November 1979, a three-phase contract was let consisting of

(1) repairing and pressure grouting the concrete pavement,
(2) placing an asphalt concrete overlay, and
(3) upgrading the existing metal beam guard fence.

Fig 1. Typical rutting and corrugation distress occurring on IH-10, Columbus.

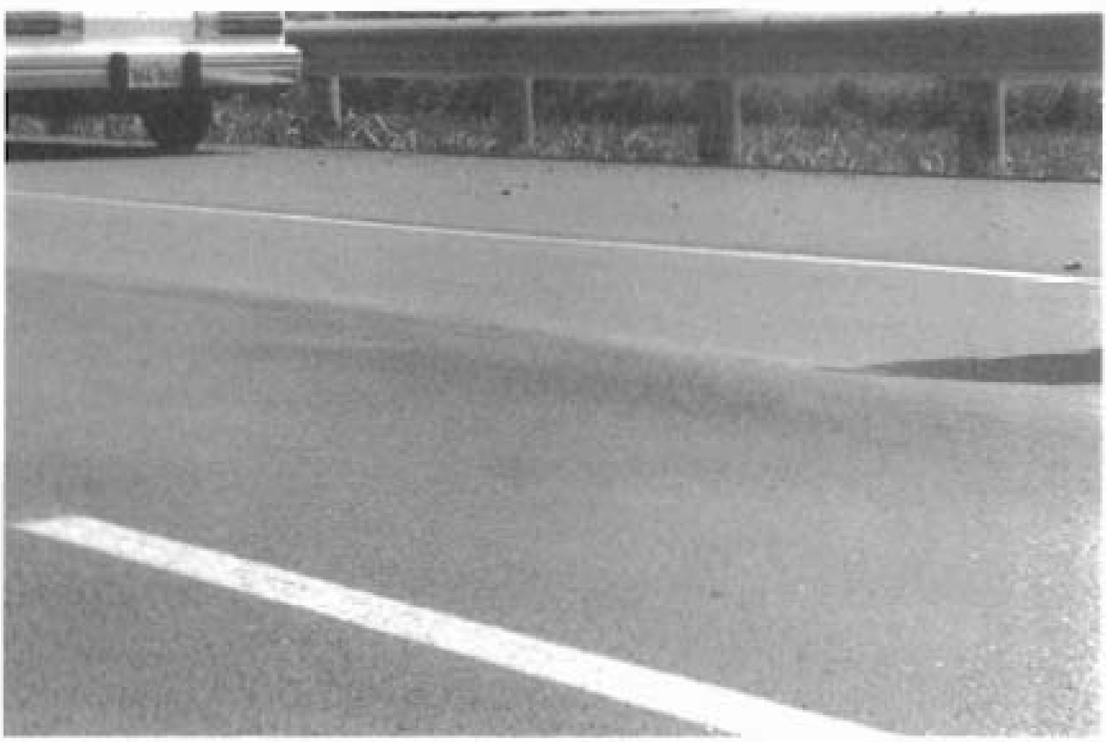

Fig 2. Typical rutting occurring on IH-10, Columbus.

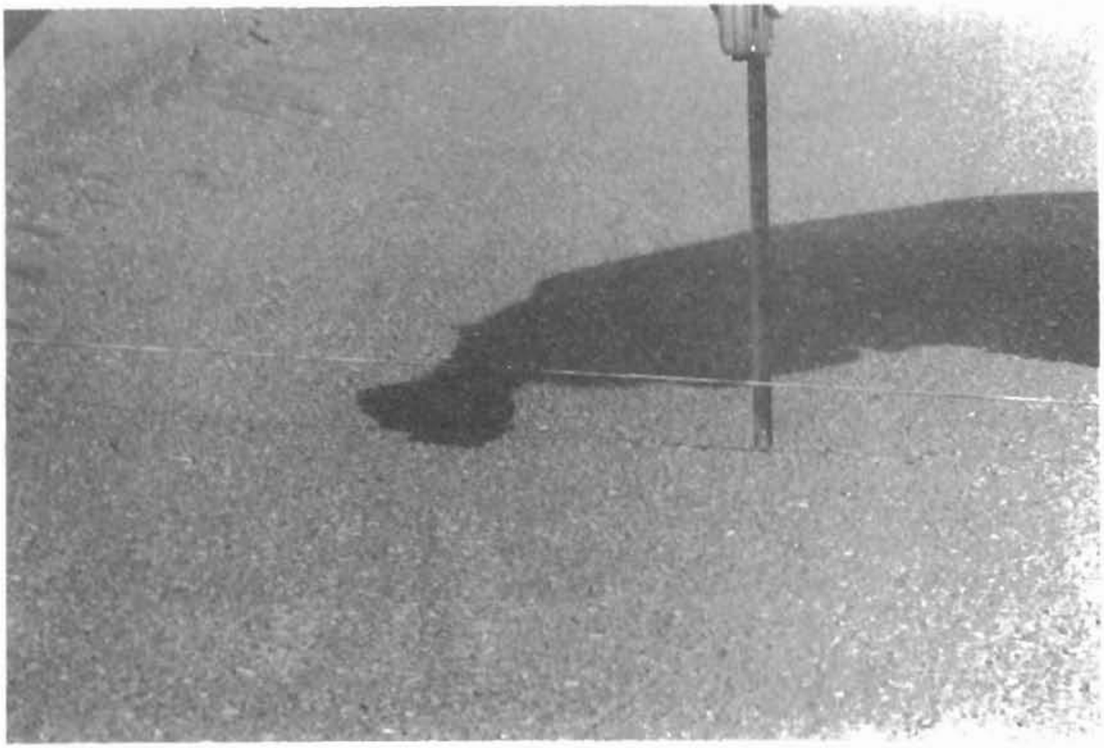

Fig 3. Magnitude of the rutting on IH-10, Columbus.

Fig 4. Planing that outlines the longitudinal variation in rutting and shoving distress on IH-10, Columbus.

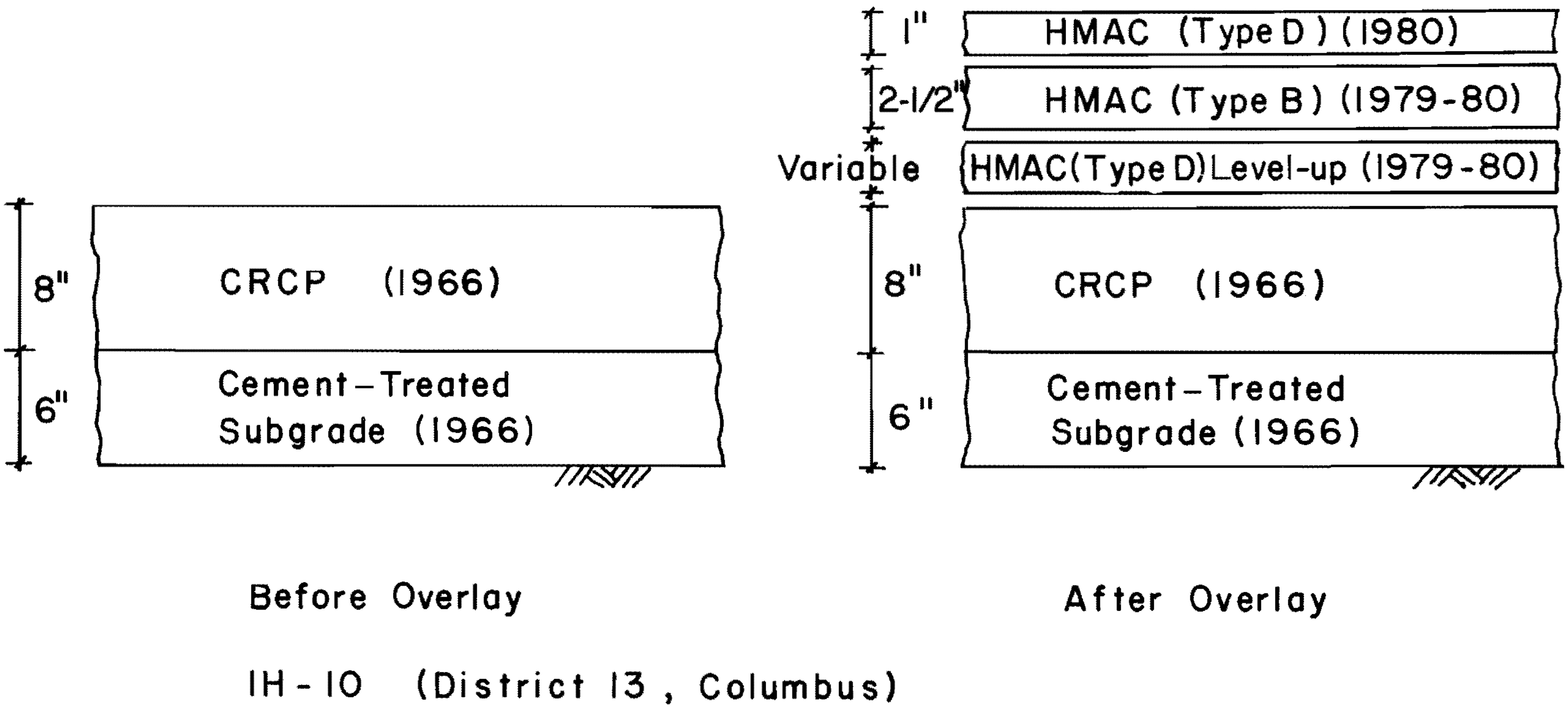

Fig 5. Existing and proposed pavement structures.

The overlay is the only portion of the contract considered in this study. The repair and pressure grouting is described in Ref 1.

Overlay operations consisted of a level-up followed by 2.5 inches of hot mix asphalt concrete base for an average depth of approximately 3 inches. A one-inch wearing course provided the final surface.

Construction of the Type D level-up course began on October 15, 1979. The level-up layer varied in thickness from about one to three inches. Placement of the level-up was completed on April 8, 1980. Appendix A1 shows the locations where the level-up course was required.

Construction of the 2.5-inch Type B base course began on October 22, 1979 at approximately the midpoint of the job and proceeded westward. Figure 6 illustrates the construction sequence. A laydown machine was used to place the mixture and tandem double drum vibratory and medium pneumatic rollers were used to compact the mixture. The eastbound and westbound lanes were laid in three mats of 15 feet, 12 feet, and 11 feet for the inside lane and shoulder, outside lane, and outside shoulder, respectively. On January 18, 1980, base course paving operations were suspended due to poor weather conditions. On March 10, 1980, paving operations resumed and continued until completion on May 5, 1980.

Construction of the 1-inch Type D wearing course began on May 6, 1980, and continued until July 1, 1980, when paving operations were halted because of the observed distress. Because the distress began in sections without the wearing course and because the distress continued to occur even in those areas with the wearing course, it was concluded that this material did not fail and did not contribute significantly to the overall failure. The remainder of the report deals with the Type B base course. The wearing course materials are described in Appendix A.

MIXTURE DESIGN

Three different mixtures were used in the overlay project, designated as designs 1, 2, and 5 (Table 1 and Fig 8). Designs 1 and 2 were similar with asphalt contents of 4.7 and 4.9 percent, respectively. Design 5 was markedly different from the other two, containing a different field sand and a much lower asphalt content of 4.0 percent. The areas in which each design was used are shown in Fig 9.

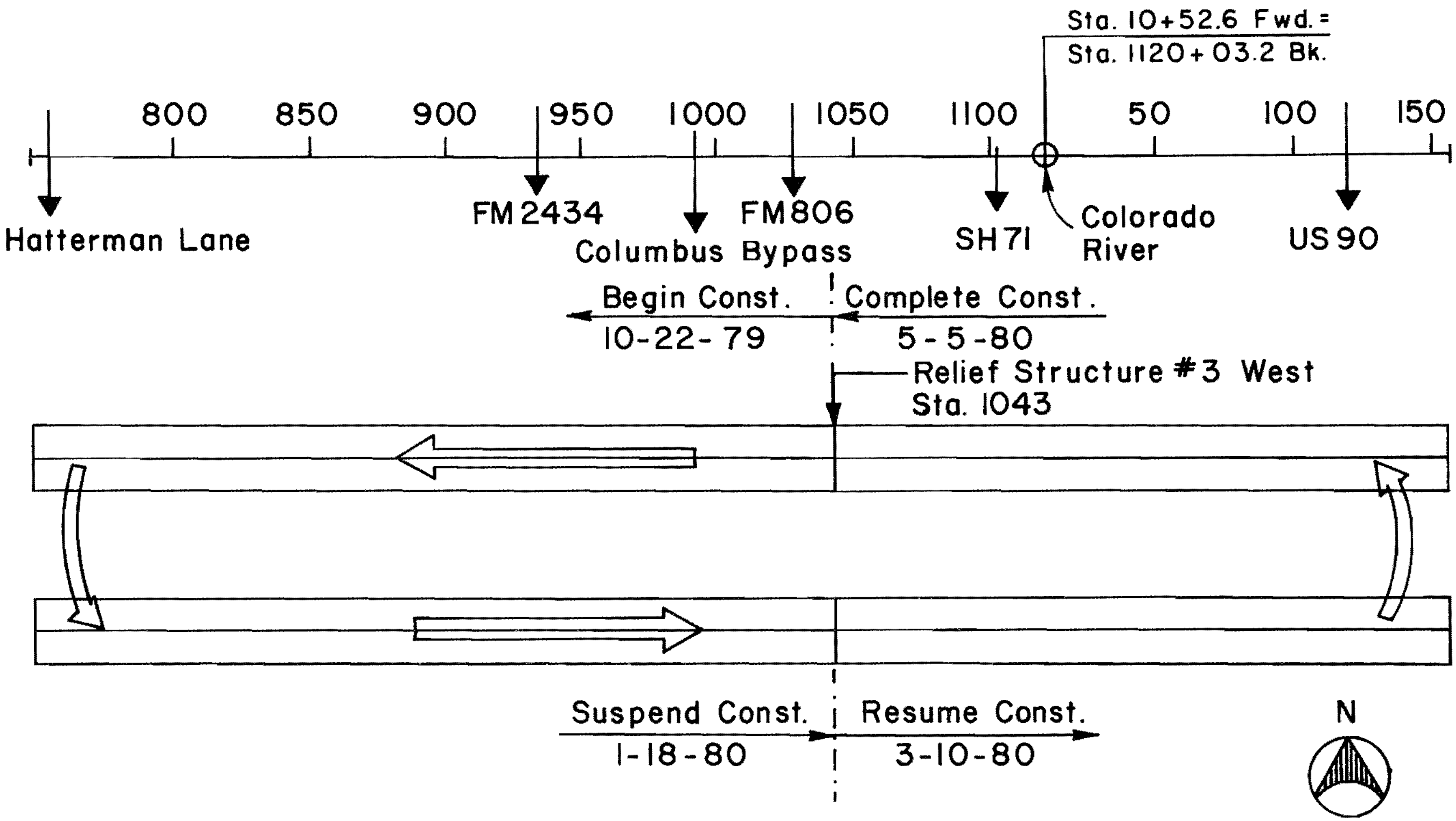

Fig 6. Construction limits and sequence for Type B base course mixture.

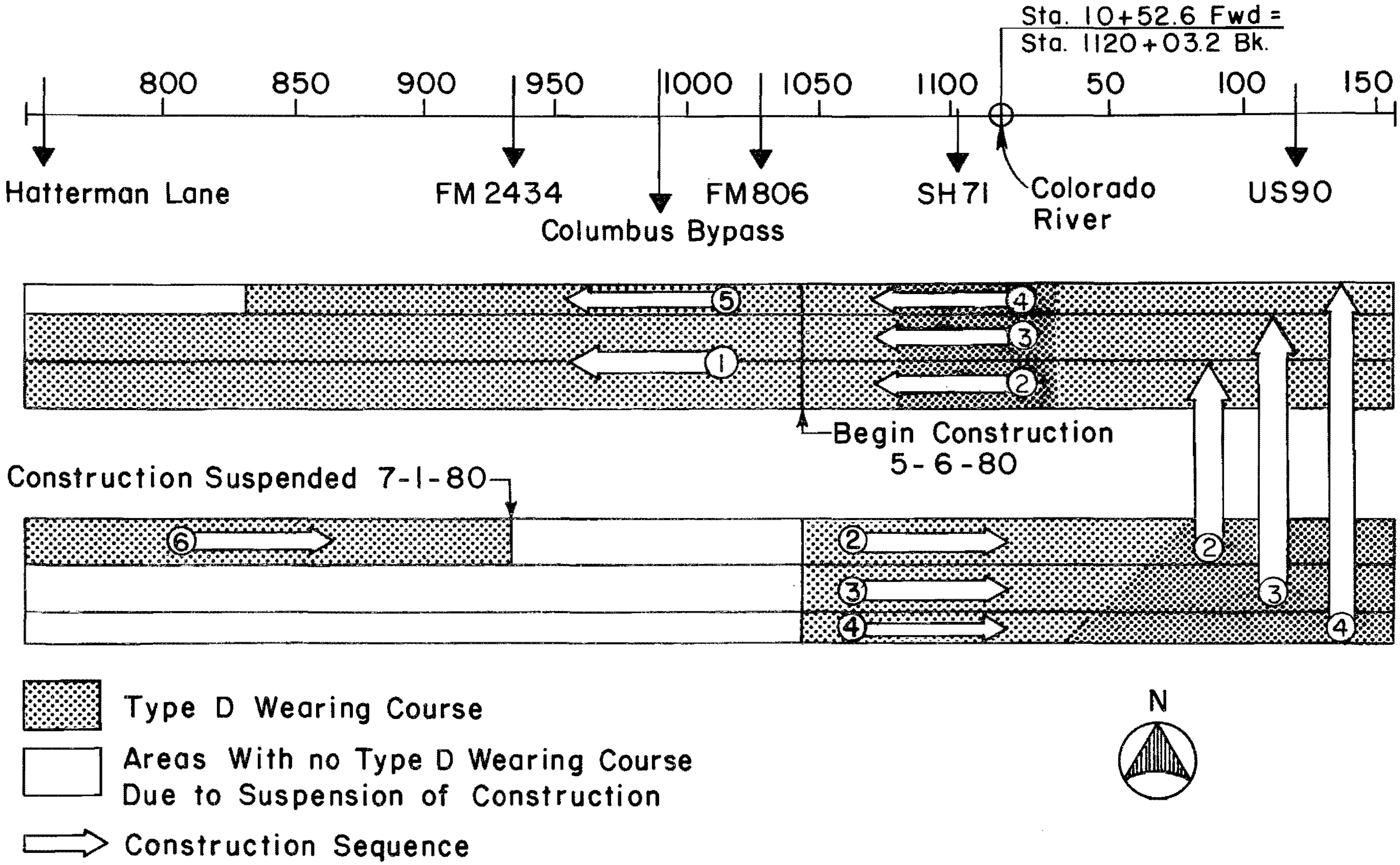

Fig 7. Construction sequence for Type D wearing course.

TABLE 1. BASE COURSE MIXTURE DESIGNS

	Percent by Weight of Total Mixture				
Design	Coarse Aggregate	Gem Sand	Coarse Sand	Field Sand	Asphalt[3] Content
1	38.1	25.7	12.4	19.1[1]	4.7
2	35.2	24.7	10.5	24.7[1]	4.9
5	35.4	25.0	10.6	25.0[2]	4.0

1 - CCMC Pit Field Sand

2 - Class Pit Field Sand

3 - 1% NEA-3 Antistripping Agent by weight of asphalt

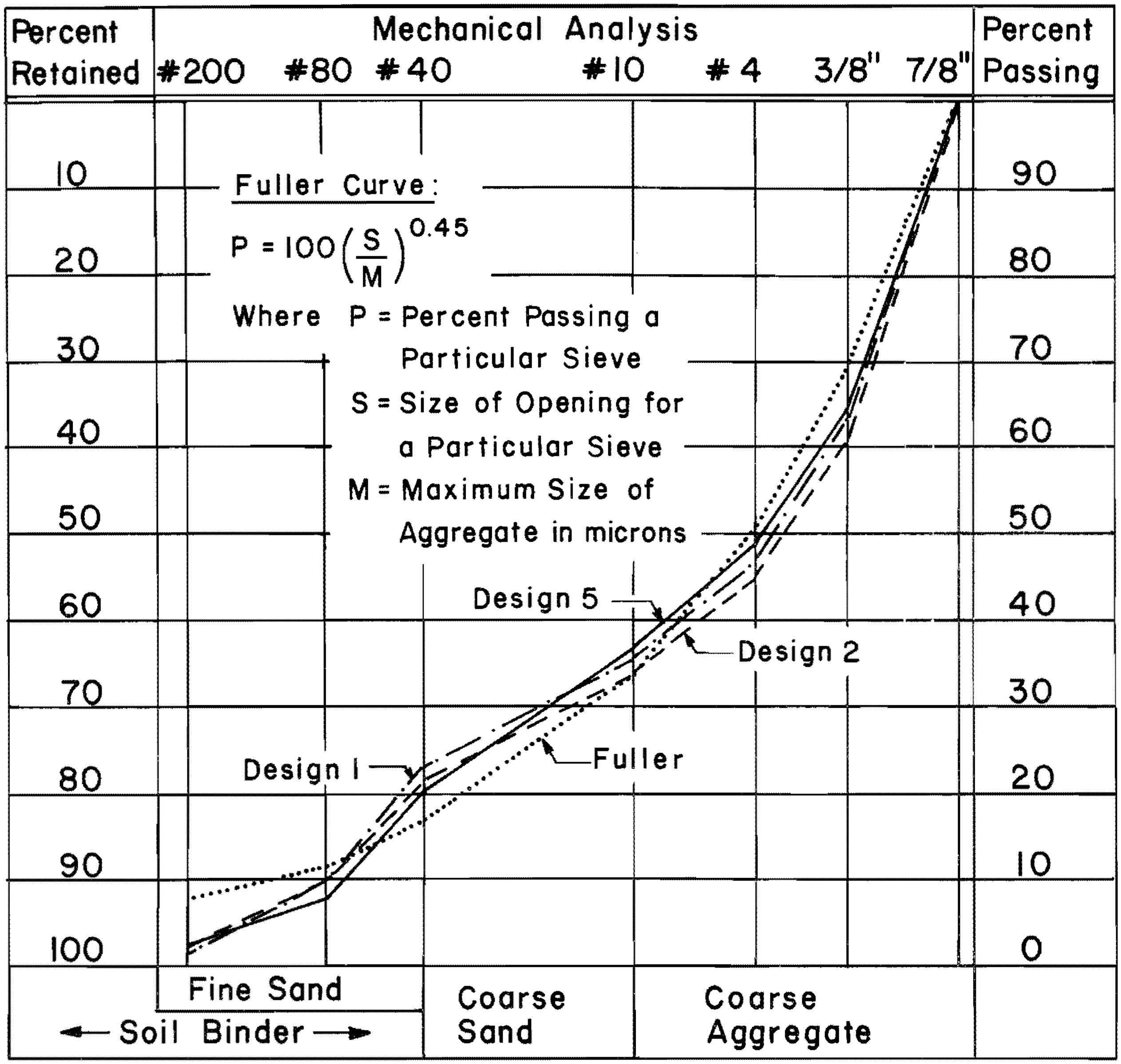

Fig 8. Comparison between Type B design gradations and Fuller gradations.

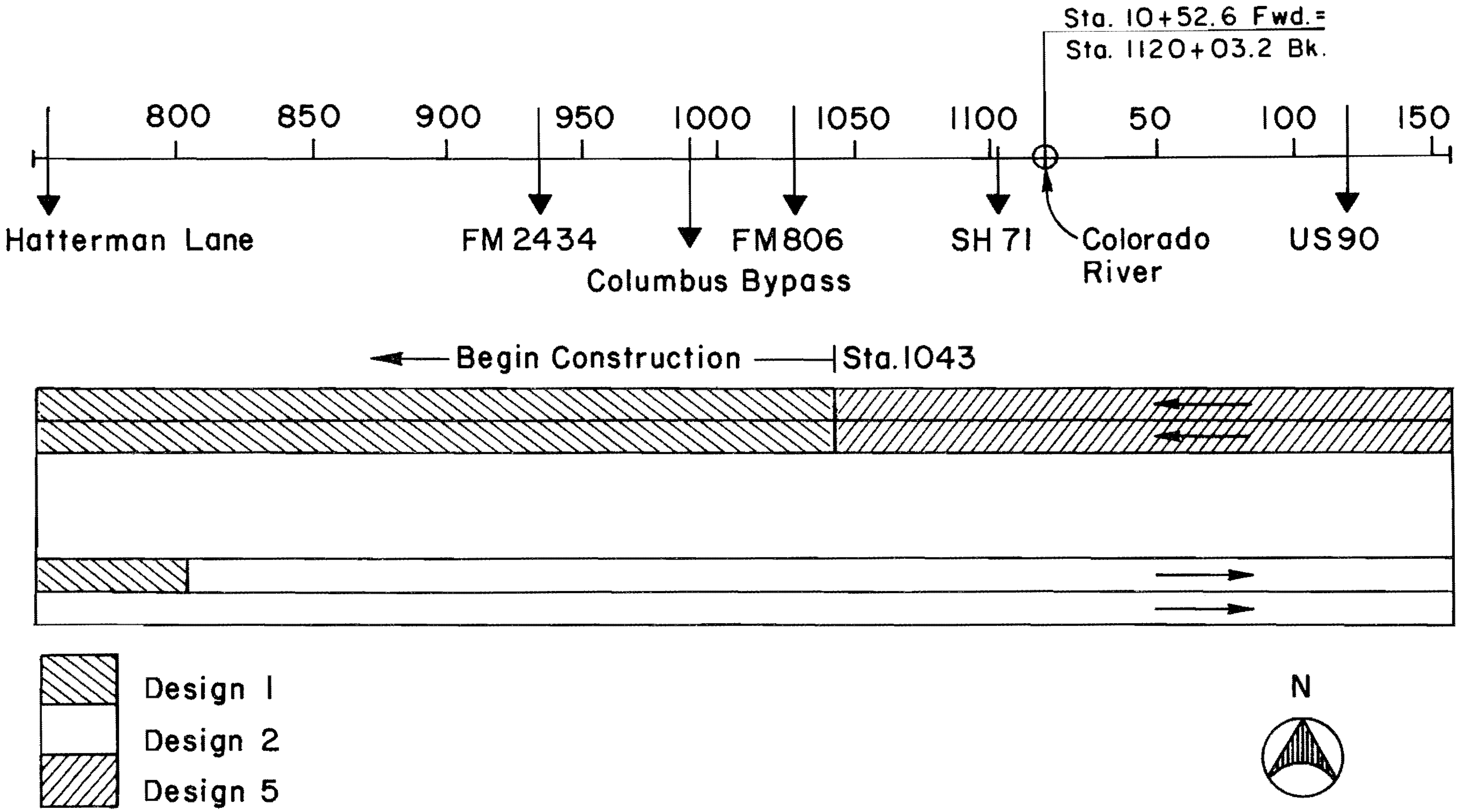

Fig 9. Locations of mixture designs.

TYPE AND SEVERITY OF DISTRESS

The types of distress were rutting, shoving, and bleeding (Figs 1-4). The results of four condition surveys conducted between October 9, 1980 and September 9, 1981 by project personnel from the Center for Transportation Research are summarized in Fig 10. Four pavement distress conditions were designated as described in Table 2. These distress conditions were (1) minor or none, (2) moderate, (3) moderate to severe, and (4) severe. In some areas of the westbound lane the severe distress was so extensive that traffic was diverted to adjacent lanes.

The progressive nature of the distress can be seen by observing that areas with initial distress designated as minor or moderate gradually became more severe. Overlay construction began on October 15, 1979, and the first evidence of distress was observed in June 1980 in the westbound outside lane. By April 1981 distress was observed in practically all sections of the project that were constructed between October 1979 and January 1980, i.e., everything west of Station 1043 (Fig 6).

STUDY APPROACH

All pertinent construction data were collected for analysis. These data were separated by design and location on the project and analyzed to determine if any patterns emerged that relate construction data to distress.

Cores from twenty-six locations (Fig 11) involving the four categories of distress were obtained. Approximately 18 cores were taken at each sampling location. In many highly distressed areas cores could not be secured because the material would not remain intact during the coring process. In such cases samples were secured as close to the area as possible. Cores were sawed into their constituent layers before testing and specimens were selected for each proposed test.

Bag samples of all materials used on the project were obtained from stockpiles and compacted in the laboratory. These specimens were then tested using a variety of laboratory test procedures to provide additional data for use in correlating test measurements to distress in an attempt to determine the probable causes for the distress.

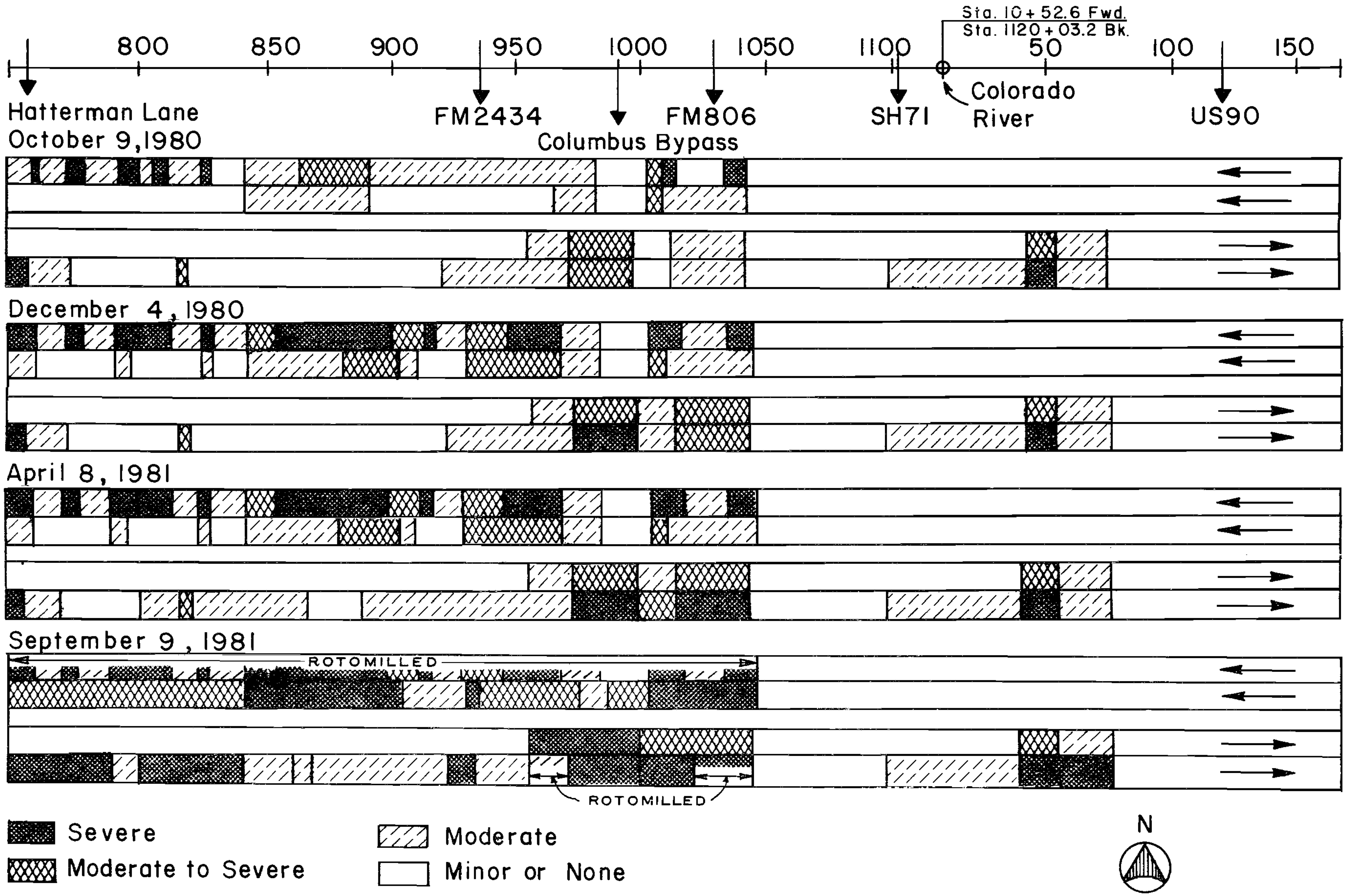

Fig 10. Condition survey.

TABLE 2. DISTRESS CONDITIONS

Distress Condition	Rut Depth, inches	Shoving
Minor to None	0 - 0.5	None
Moderate	0.5 - 1	None
Moderate to Severe	1 - 2.5	Slight
Severe	1 - 2.5	Pronounced

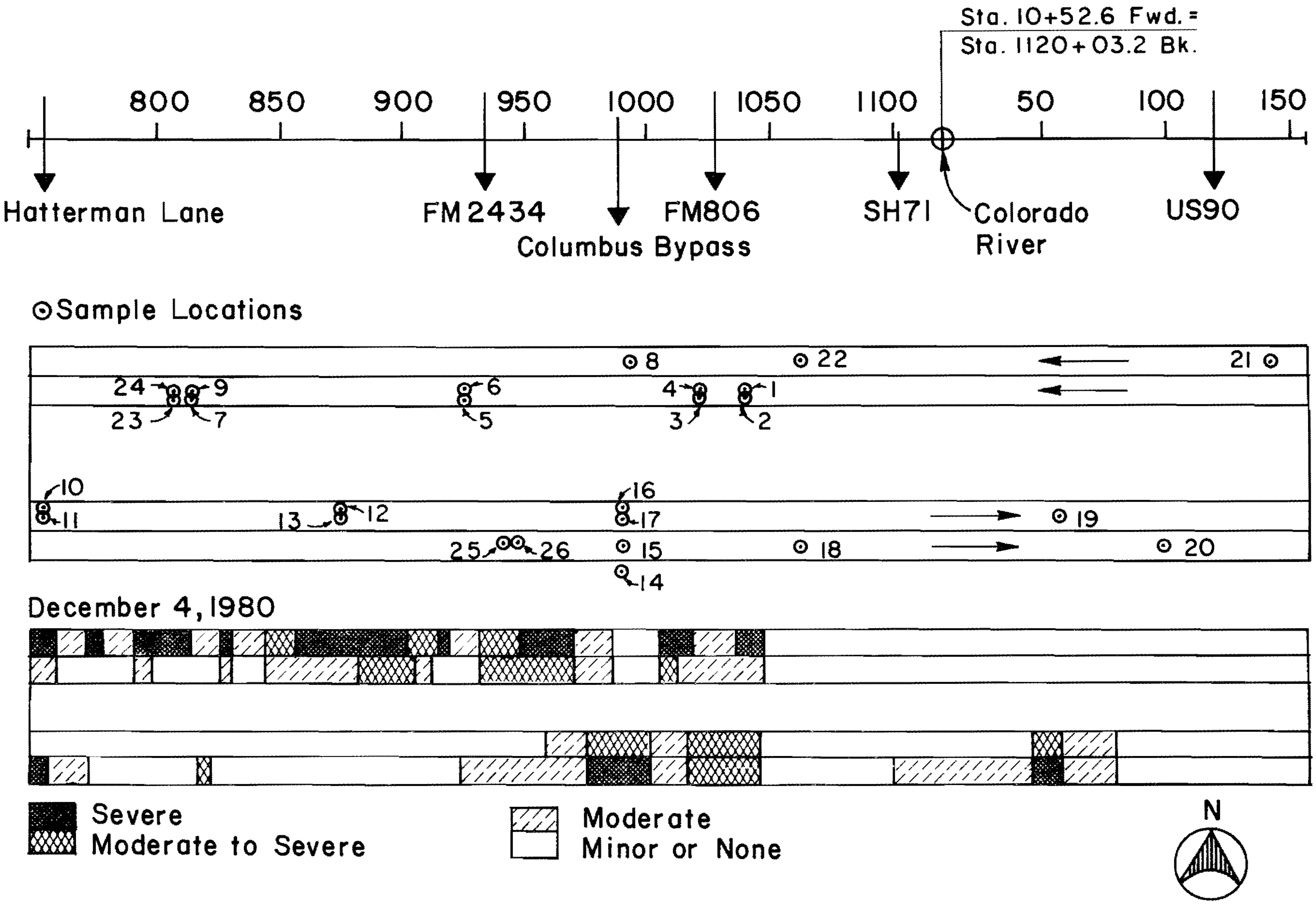

Fig 11. Sampling locations and corresponding condition survey.

CHAPTER 2. ANALYSIS AND EVALUATION

As shown in the condition surveys (Fig 10), the distress, which was due to the base course, was concentrated in designs 1 and 2 with the major distress beginning in design 1. No major distress occurred in design 5, although an extremely small amount of rutting was visible at the time of the last condition survey. As previously noted, designs 1 and 2 were similar (Table 1 and Fig 8), with design 2 having only 0.2 percent more asphalt, while design 5 was markedly different, containing a different field sand and asphalt content.

CONSTRUCTION SEQUENCE

All of design 1 and a large portion of design 2 for the Type B base course layer were placed prior to January 18, 1980, at which time construction was suspended because the weather was too cold and wet for satisfactory placement. The remainder of design 2 and all of design 5 were placed after construction resumed on March 10, 1980. The Type B layer served as a temporary surface and was exposed to the action of both the environment and traffic until it was covered by the Type D wearing surface. Construction of the wearing surface began May 6, 1980. The sequence of construction is shown in Fig 7 and indicates that design 5 and the later portion of design 2 were covered first.

A review of the condition surveys (Fig 10) indicates that the majority of the distress occurred in those sections which were exposed during the winter months of 1979-1980. In addition, distress in these exposed areas occurred first in those sections which were laid first, i.e., the ones which were exposed for the longest period of time.

ASPHALT CONTENT

There were definite asphalt characteristics associated with the various mixture designs and the mixtures were definitely related to distress. Figure 12 summarizes the relationship between extracted asphalt contents determined at the residency laboratory during construction and distress for each day of construction. No definite relationship between asphalt distress was evident within a particular design. However, the extracted asphalt contents for design 1 were generally above the design value; for design 2 they were about equal to the design value; and for design 5 they were essentially below the design value. Major distress first began in design 1, which had the higher asphalt contents, and later in design 2, which contained essentially the design asphalt content. Essentially no distress occurred in the under-asphalted design 5.

The variation in extracted asphalt contents for each mixture design, as determined in the residency laboratory, is shown in Fig 13. Approximately 17 percent of the report asphalt contents were outside of the maximum tolerance of 0.5 percent (Ref 2).* Nevertheless, there was no direct relationship between distress and asphalt content for the various designs.

Of particular importance is the fact that there is an apparent significant and consistent discrepancy between the extracted asphalt contents obtained from roadway slabs by D-9 and those obtained by the residency from samples taken from the plant during construction (Fig 14). In almost every case the asphalt contents of slabs reported by D-9 were larger than those reported by the Columbus residency for the same day of construction. Consistent differences of this magnitude could hardly be due to random differences in material or test procedure. In addition, plant records indicated that the correct amount of asphalt was added.

While the difference cannot be explained, two possible causes, which involve the testing and sampling procedure, are evident. Examination of the extraction test procedure used by the residency and D-9 suggested that a possible cause could relate to the ashing procedure. The procedure used by D-9 involved the use of an oven capable of producing temperatures in excess of $1400^{\circ}F$. The residency laboratory,

*The standard deviations of the asphalt contents were essentially equal to the tolerance.

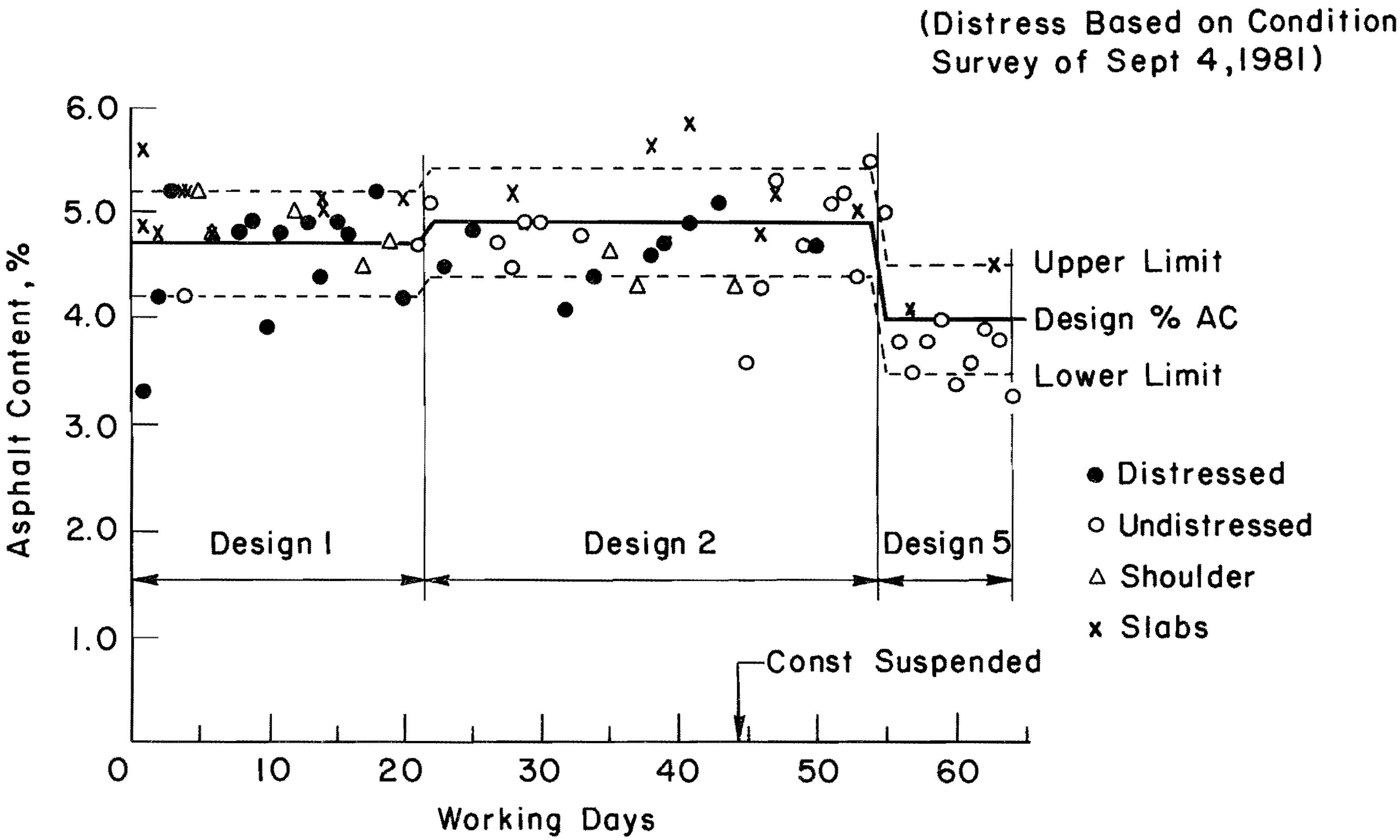

Fig 12. Relationship between extracted asphalt content and distress during construction.

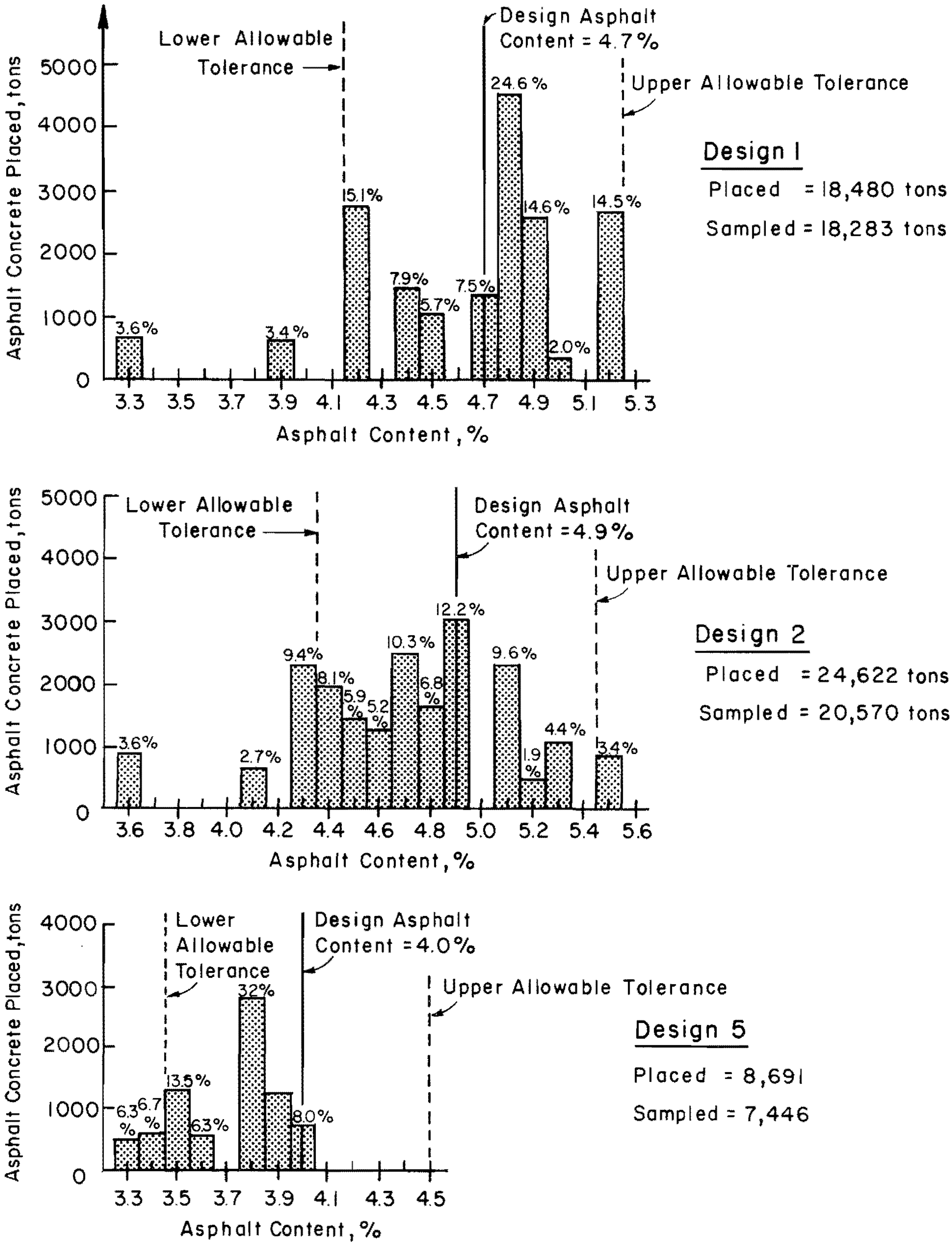

Fig 13. Asphalt variations, job control extractions.

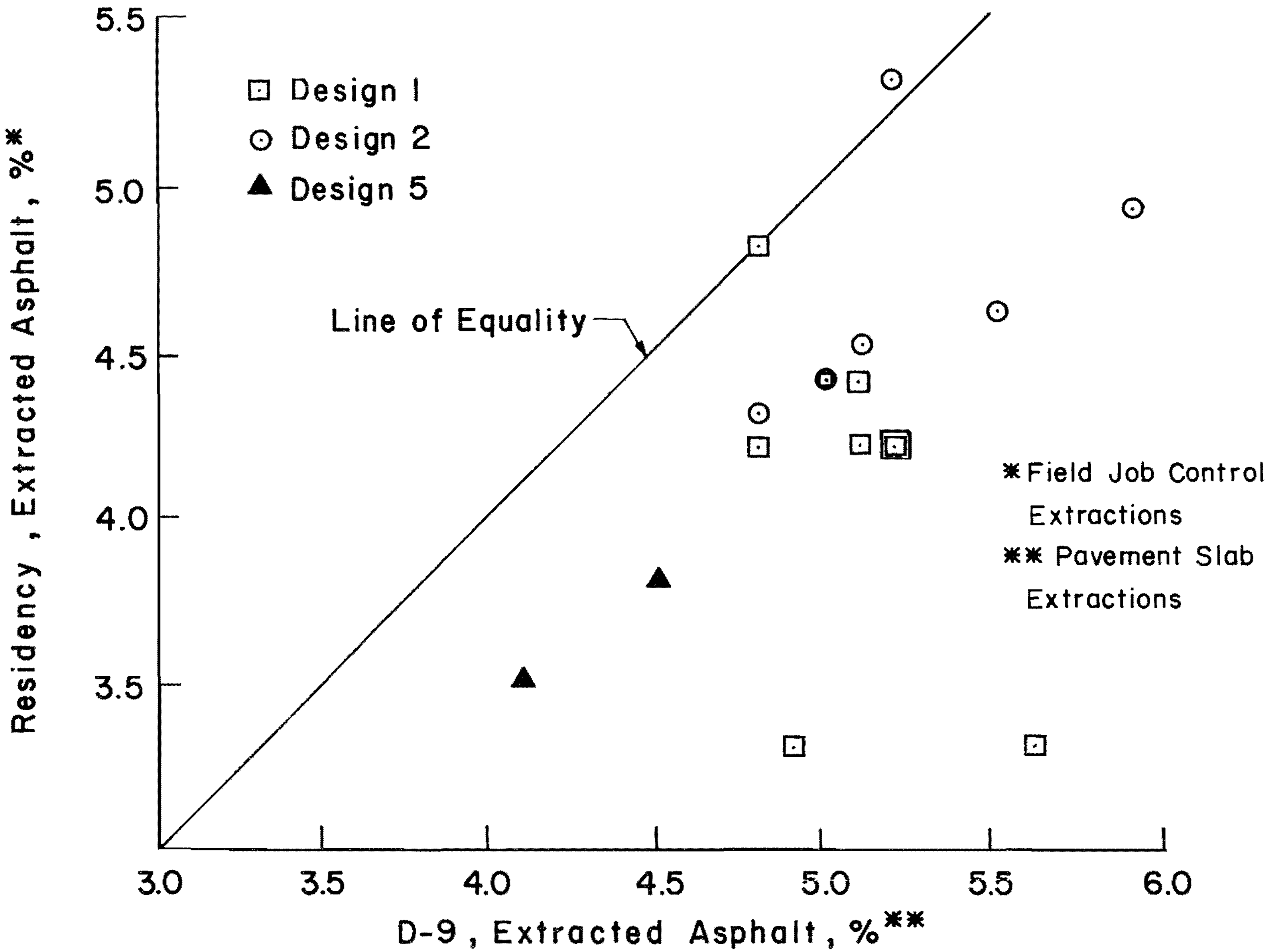

Fig 14. Comparison between Residency and D-9 extractions.

however, used a Bunson burner which possibly would not produce enough heat in a short period of time to cause efficient and complete ashing. The second possibility is the fact that the slabs which were sent to D-9 were from the more distressed areas and with stripping beginning at the bottom of the base course a loss of some material could have resulted in an apparent increase in asphalt content. Finally, since the D-9 slabs were from the distressed areas, these slabs could possibly have had higher asphalt contents.

When project personnel, as a part of the study, sent duplicate samples to D-9, District 13, and the Columbus residency, all three reported similar asphalt contents. If the test procedure was the problem, these results suggest that the techniques used by all three laboratories are capable of producing similar values but that the results may be very sensitive to small differences in technique.

Although no corrections in asphalt content were made during construction as a result of the residency test values, the differences must be considered. Since the data shown in Fig 13 are based on the residency values obtained during construction, it is possible that these values are from 0.5 to 0.9 of a percent below the actual asphalt content. If this is true, then design 1 and, to a lesser extent, design 2 had an excess of asphalt, while design 5 was at or only slightly above the design asphalt content. The high asphalt contents would have produced an unstable mixture resulting in rutting and shoving.

The above would also be true if higher asphalt contents occurred during construction because of plant variations. However, plant records consistently show the proper amount of asphalt and extraction records do not indicate a large percentage of samples with high asphalt contents (Fig 12). However, if the slabs taken from distressed areas actually had asphalt contents 0.5 to 0.9 of a percent larger than the other areas, it is possible there were very localized areas with higher asphalt contents, thus contributing to failure. Figures 12 and 13 indicate that more samples in designs 1 and 2 had asphalt contents above the design value.

Using the daily extraction values and known specific gravities of all mixture constituents, the voids in the compacted mineral aggregate (VMA) and percent voids filled with asphalt (VF) were computed. No differences in VMA were detected for the various mixture designs or distress conditions. In addition, throughout the project the VMA ranged from 14.7 to 19.1 percent and averaged 17.4 percent, which compares favorably with the minimum of 13.5

percent recommended by the Asphalt Institute (Ref 3). Similarly, no differences in VF were detected for the various mixture designs or distress conditions. Monismith (Ref 4) has suggested that a VF range of 65 to 75 percent for base courses and 75 to 85 percent for surface courses is required in order to produce mixtures that perform satisfactorily in the field. The VF for these three designs ranged from 49.6 to 64.8 percent and averaged 57.5 percent, which is below the recommended values. These values are based on extracted asphalt contents which may have been lower than the actual asphalt content. An analysis of the mixtures assuming asphalt contents which were 0.5 to 0.9 percentage points higher indicated that the percent voids filled was adequate.

GRADATION

A comparison between the design gradation and the Fuller gradation, which produces a well-graded dense mixture, is shown in Fig 8. All three mixture designs had more material retained on the No. 4 sieve than the Fuller gradation. All three mixture designs are deficient in material passing the No. 200 sieve. The course sand is basically a material contained between the Nos. 10 and 40 sieves. While gradation was not the apparent cause of the observed distress, it is felt that gradation and its effect need to be considered and evaluated with respect to performance.

As with asphalt contents, differences were detected between gradations obtained by D-9 and gradations obtained by the residency. The gradations obtained from the slab samples supplied by the project to D-9 were finer than the gradations obtained by the residency during construction as indicated by the material passing both the No. 10 and the No. 200 sieves (Fig 15). Again, incomplete ashing at the residency laboratory may possibly account for the smaller amount of material passing the No. 200 sieve which was reported by the residency. The overall cause, however, is not apparent but is obviously consistent.

Using the duplicate samples obtained from pavement slabs which were used to compare extracted asphalt concrete, a comparison was made of the gradations obtained by the residency, district, and D-9 laboratories. As shown in Fig 16, the results from the district and the residency are essentially the same. The gradations reported by D-9, however, were different for two samples, with one being finer and the other coarser.

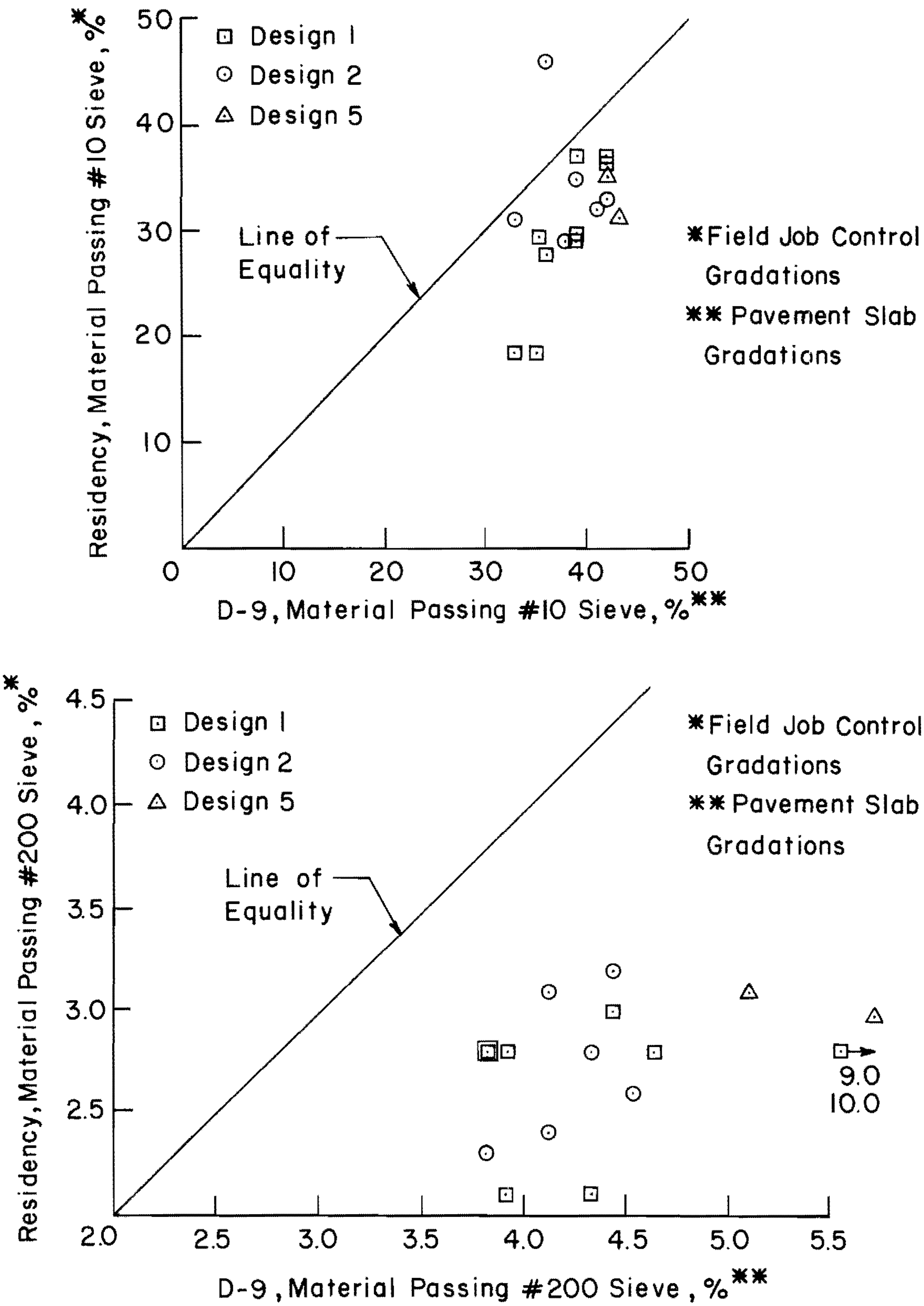

Fig 15. Comparison between Residency and D-9 gradations.

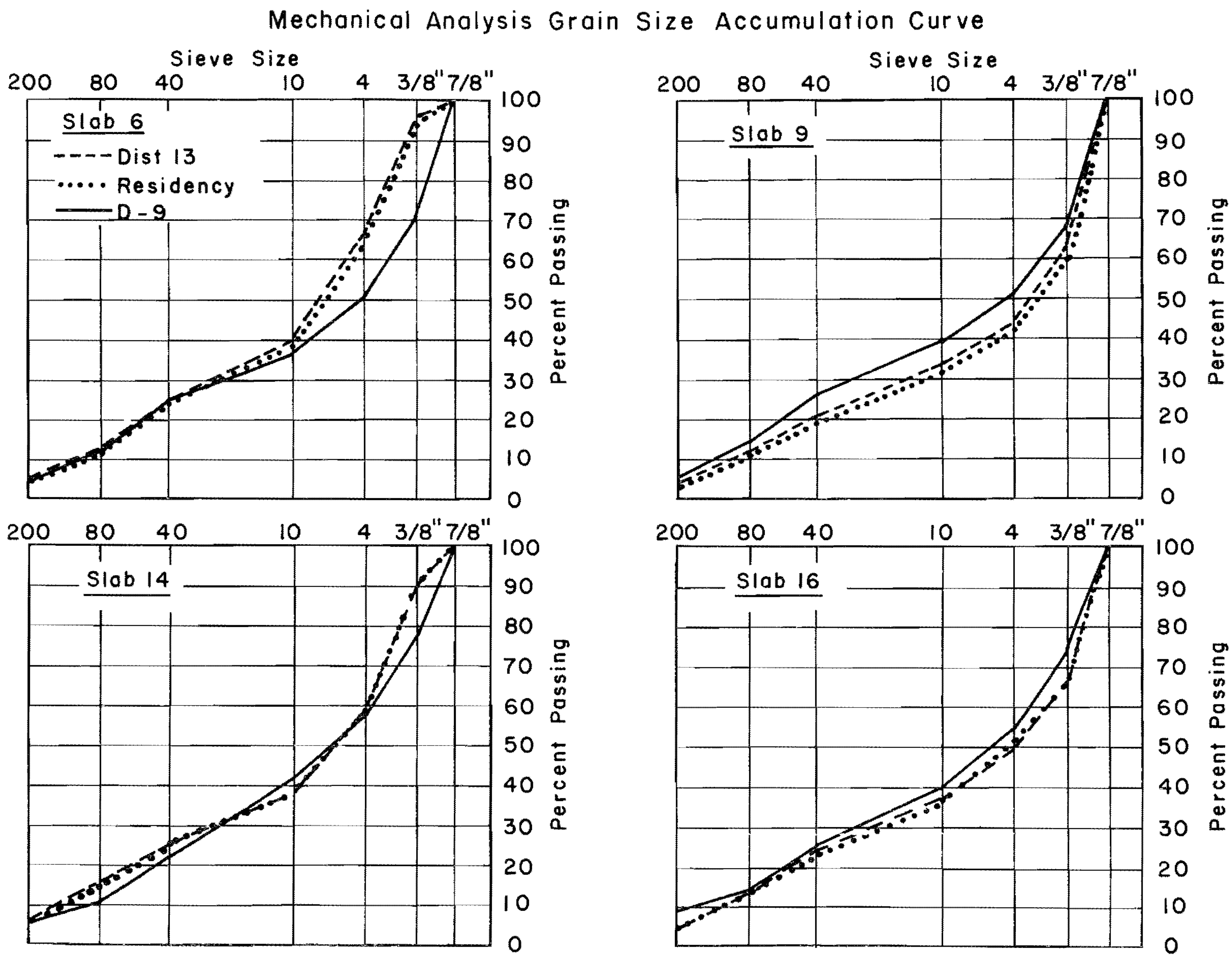

Fig 16. Comparison of gradation results from D-9, District 13, and the Residency on four pavement slabs.

VISCOSITY

Penetration and viscosity tests were performed on the asphalt cements recovered from the Abson process. No significant differences in penetration or viscosity of the residual bitumen were found for the three mixture designs. In addition, no differences were detected in penetration or viscosity for materials secured from either the distressed or nondistressed areas. Therefore, it is concluded that the asphalt cement properties themselves did not vary significantly along the job and were probably not a major factor contributing to these premature failures.

SAND EQUIVALENT

The average and range of the sand equivalent values by design and category of distress as computed by the residency laboratory during construction are shown in Fig 17. Designs 1 and 2 contained CCMC sand and design 5 contained Class sand. The average sand equivalents for the CCMC and Class sand were 45 and 49, respectively. Although these mean values were above the minimum acceptable value of 45 (Ref 2), 50 percent of the tests on the mixtures containing CCMC sand (designs 1 and 2) were below 45 with values as low as 32 and as high as 59, indicating a high degree of variability in this sand (Fig 17). For the Class sand only 25 percent were below 45 with values ranging from 40 to 60.

During construction the contractor was required to change to the Class pit because of difficulty with localized areas of dirty sand. Although the above suggests that the observed distress could quite possibly be related to the sand, no definite relationship was found between the average sand equivalent values and observed distress for designs 1 and 2. In addition, inspection of the individual sand grains under a microscope indicated that both sands had essentially the same shape. Thus, it is felt that the sand was not a direct cause of the distress unless the variation in fines in designs 1 and 2 could have caused problems.

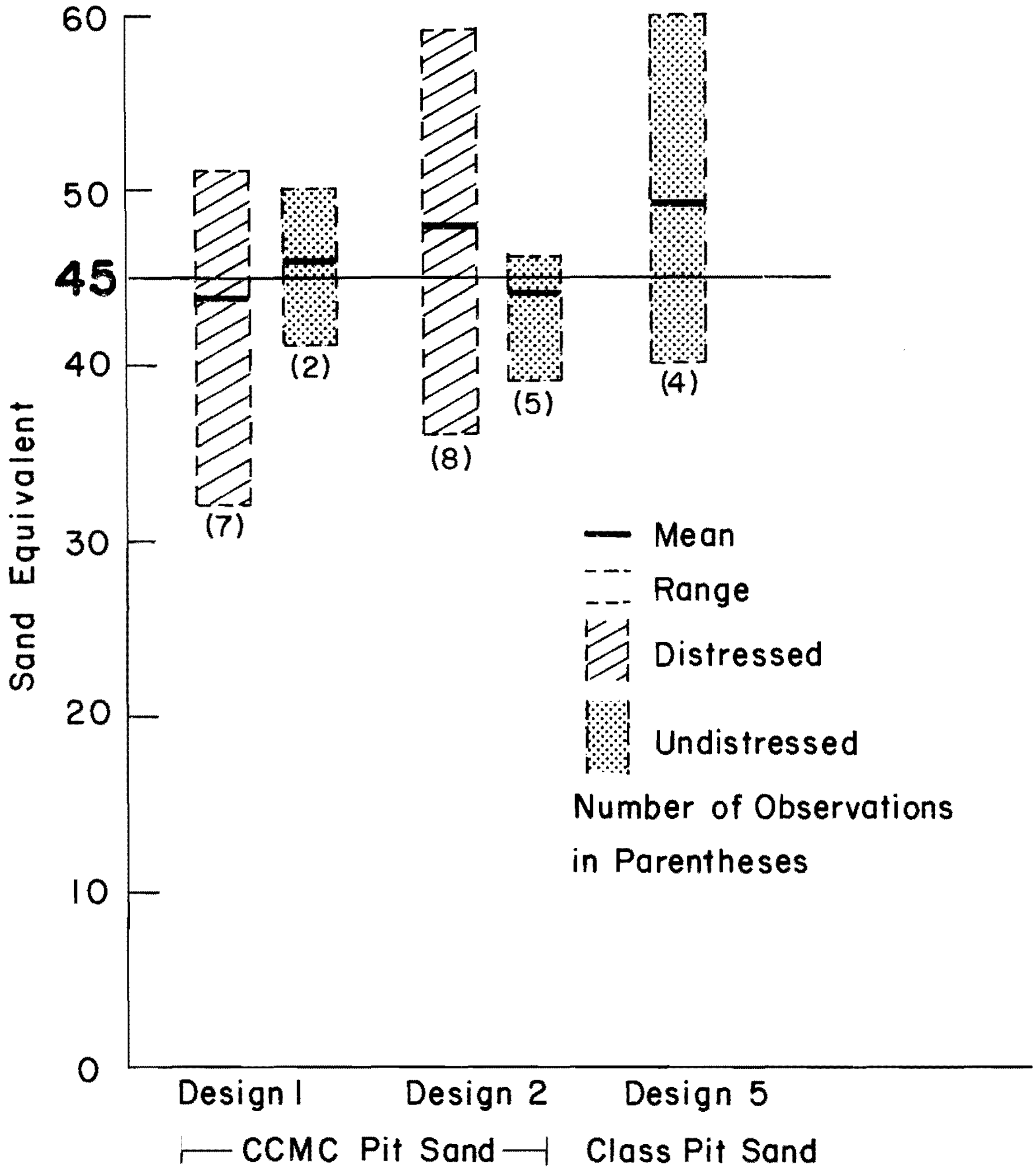

Fig 17. Sand equivalent values.

DENSITY

Figure 18 shows the density of cores and laboratory prepared Hveem stability specimens for each day of construction along with the densities associated with 100, 97, and 95 percent of theoretical maximum density.* It can be seen that the densities of design 1 mixtures were always less than 97 percent of theoretical maximum density, design 2 densities were slightly greater than 97 percent, and design 5 densities were all much greater than 97 percent. In addition, as shown in Fig 18, the densities for designs 2 and 5 began to increase as soon as construction resumed. Although no satisfactory explanation is available, this behavior suggests a change in compaction conditions or materials.

Densities for each core were determined by project personnel. Average core densities and the range of values for the three designs are shown in Figs 18 and 19. Average core density for design 5 (150.9 pcf) was significantly higher than the overall average core density for the undistressed areas of either designs 1 or 2 (146.5 and 146.2 pcf, respectively). A comparison of the core values and Hveem specimens suggests that the actual densities are correct. The values for design 5, however, appear to be quite large and it should be noted that these densities could not be duplicated in the laboratory.

A laboratory compaction study was performed to simulate field density (cores), gradation, asphalt content, and compaction temperature (residency records) on mixture properties. This study is described in detail in Appendix B. Specimens were compacted to densities typical of those found in designs 2 and 5 for both distressed and undistressed locations. None of the data collected adequately explain the differences in density observed between designs 2 and 5. As noted, the design 5 density of 151.5 pcf could not be obtained in the laboratory with the gyratory shear compactor.

Based on the data and the observed behavior, it is felt that while the actual values of density and the observed trends are correct, the value of relative compaction may be too high.

STABILITY

Figures 20 and 21 show the Hveem stability of laboratory specimens prepared daily during construction. Most of these values were between 35 and 45 and thus met the minimum stability requirement of 30 (Ref 2). There was no apparent relationship between stability and distress within designs 1 and 2. However,

*Based on specific gravities obtained by District 13

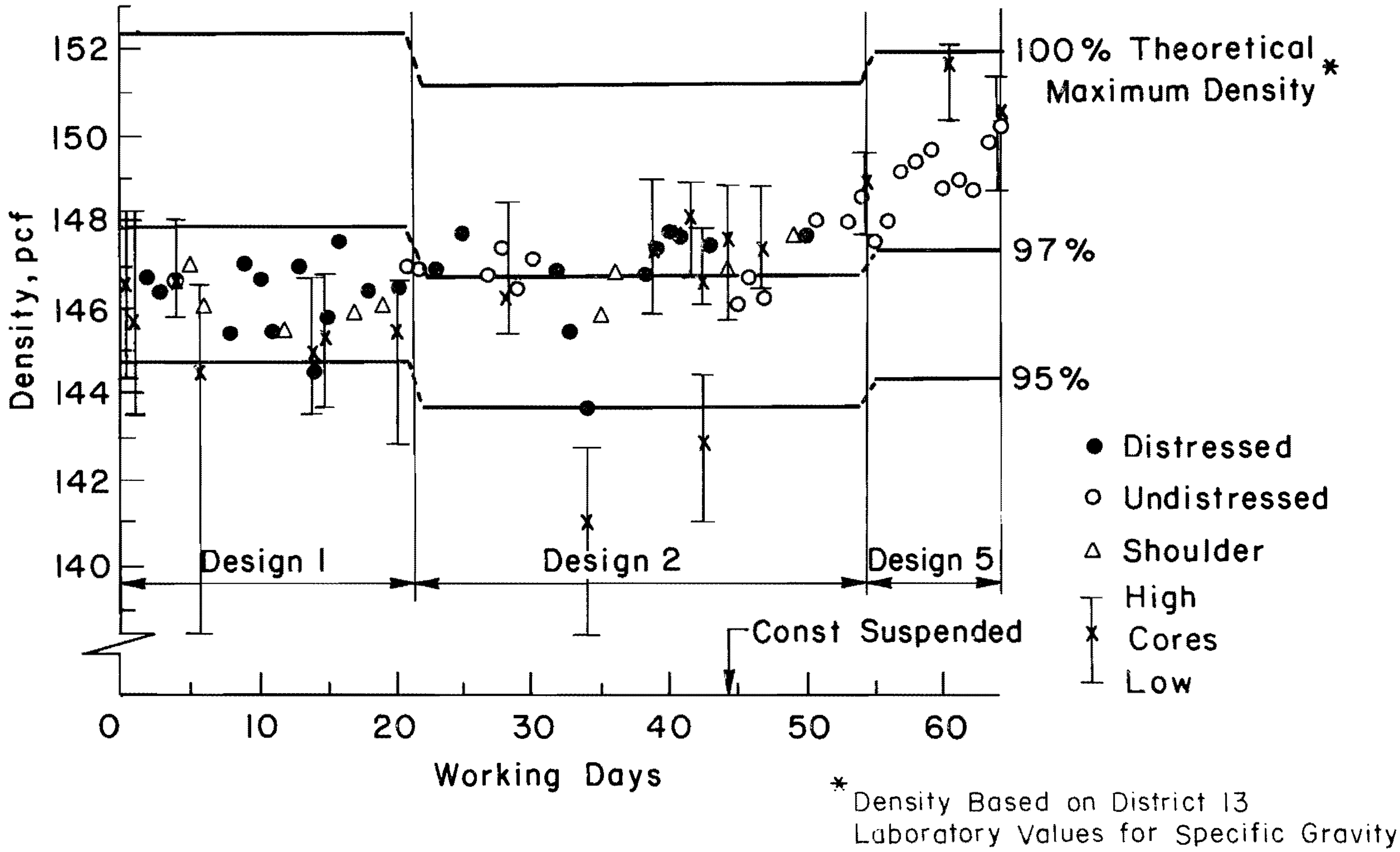

Fig 18. Density of job control specimens and cores.

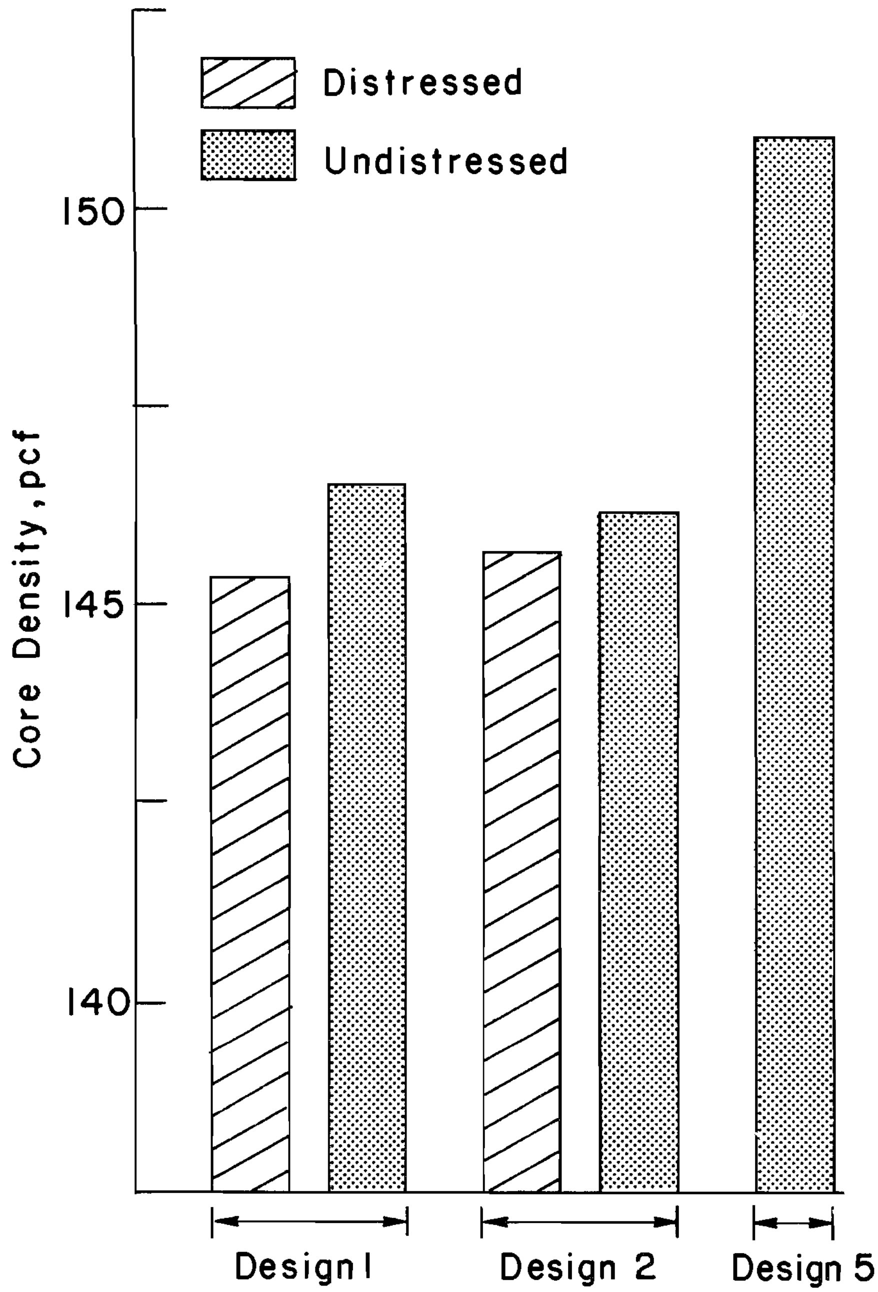

Fig 19. Core densities for the various mixture designs.

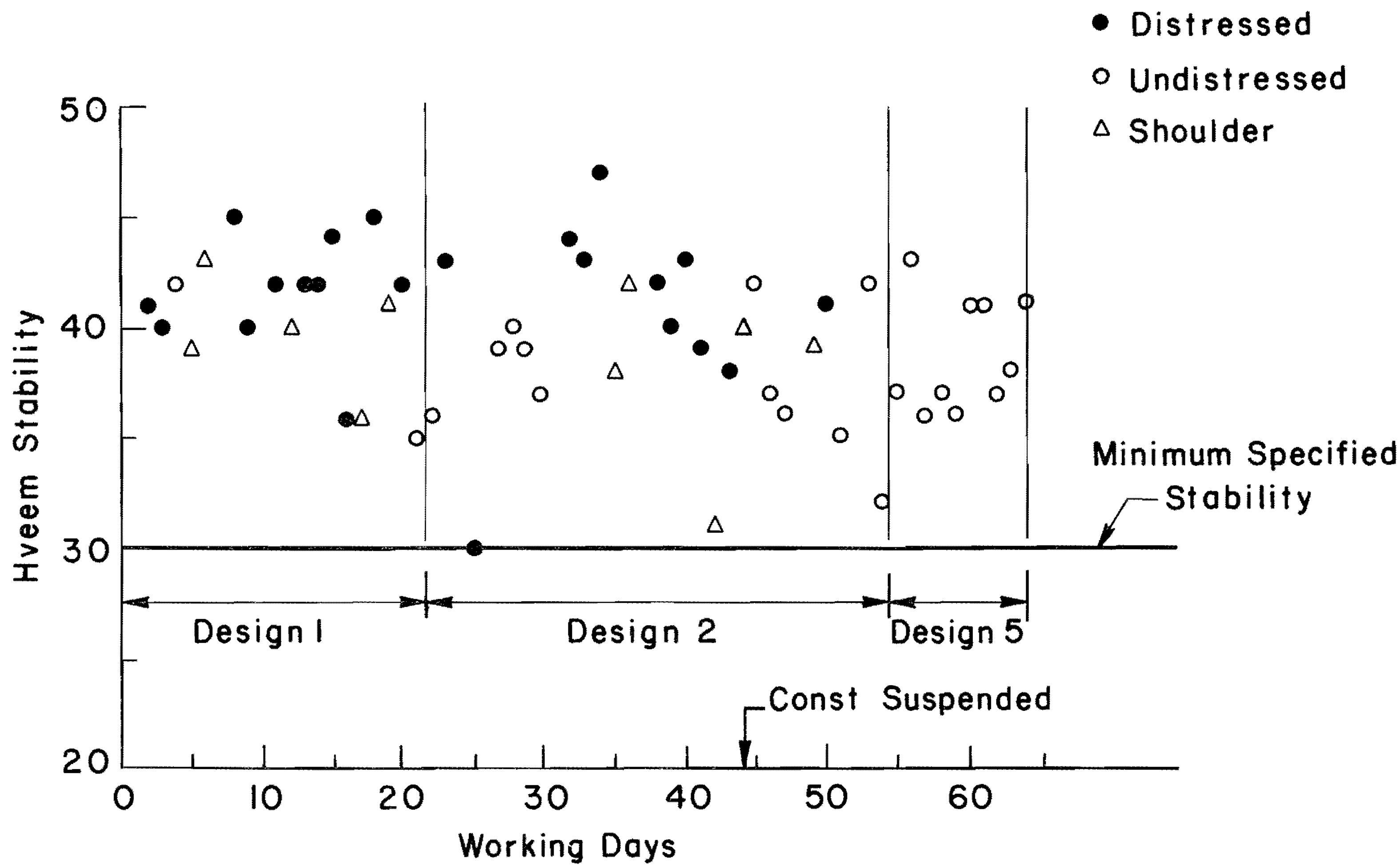

Fig 20. Hveem stability of job control specimens.

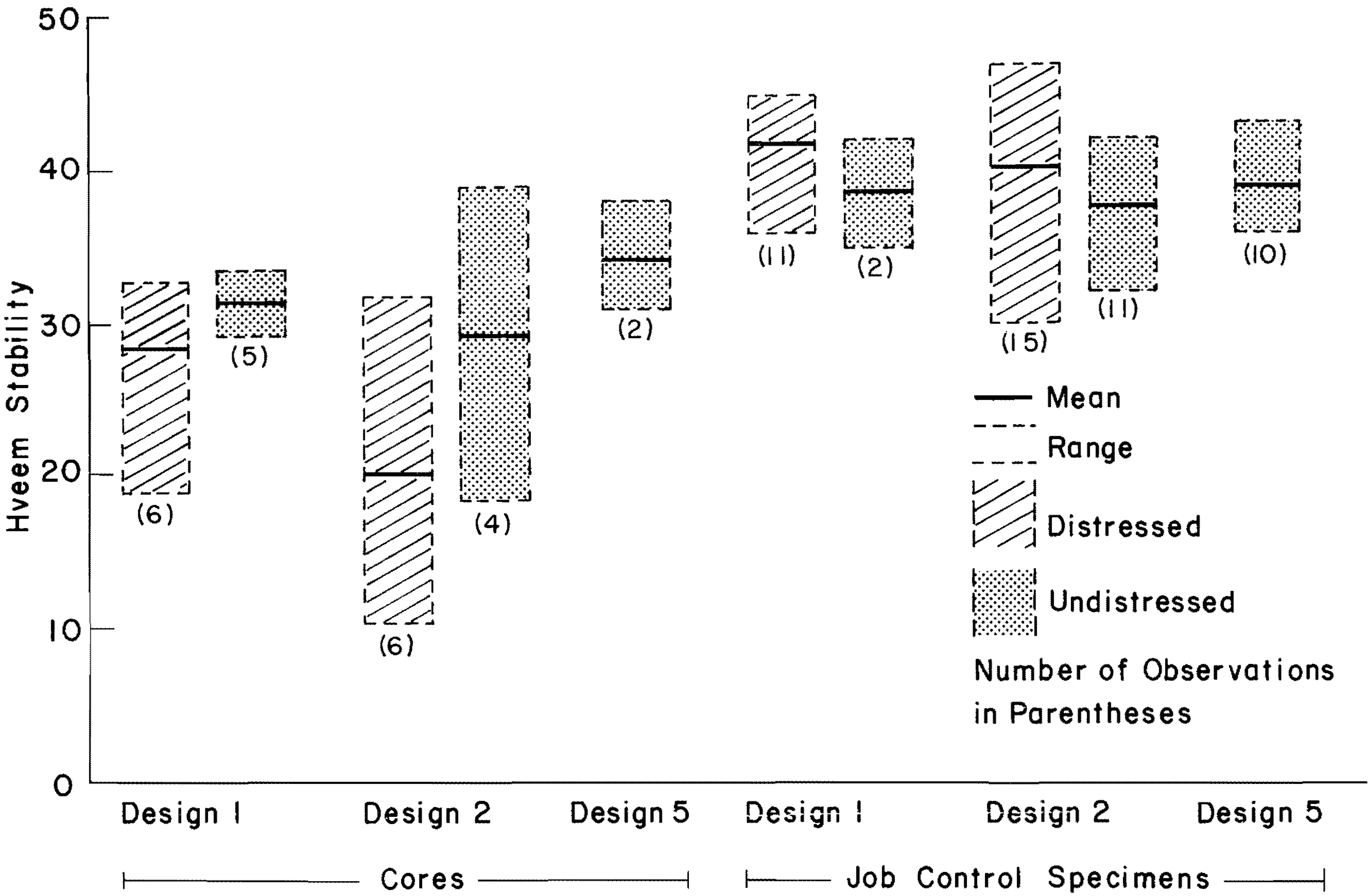

Fig 21. Hveem stability of cores and job control specimens.

more distress was exhibited in design 1 materials than in design 2, and design 5 showed no distress.

Hveem stabilities were also determined for two or three cores at most sample locations and the results are summarized in Fig 21. It should be noted that these Hveem stability values cannot be interpreted in terms of specifications since the tests were on cores; however, they can be evaluated on a comparative basis. The results indicate that the average stability for design 5 was higher than that for either design 1 or design 2, but only two cores were tested for design 5. Stabilities were generally lower in the distressed areas than in the undistressed areas and the variation tended to be slightly smaller. Many of the values were extremely small, ranging as low as 10.

Hveem stabilities from the laboratory compaction study mentioned above are of the same general magnitude as those reported by the residency as a part of the daily construction records (Fig 21). This is true for the standard compaction for designs 2 and 5 and for specimens prepared at target field densities for design 2. Since the field density of 151.5 pcf for design 5 could not be achieved, no test results were available for design 5.

While the mixtures on this project satisfied the commonly required Hveem stabilities of 30 and actually ranged from 35 to 45, it is still recommended that the required stability value be increased to 35 or 40 for high volume highways to help insure that stability problems are minimized.

TENSILE PROPERTIES

Tensile strengths were determined for cores taken from the roadway. These values are summarized in Fig 22. Static and resilient moduli of elasticity were also measured and showed the same trends as tensile strength. The dry tensile strengths for distressed and undistressed areas were not significantly different for designs 1 and 2. However, design 5 had a higher overall average dry tensile strength than either design 1 or design 2. Wet tensile strengths were also evaluated and are discussed under moisture susceptibility.

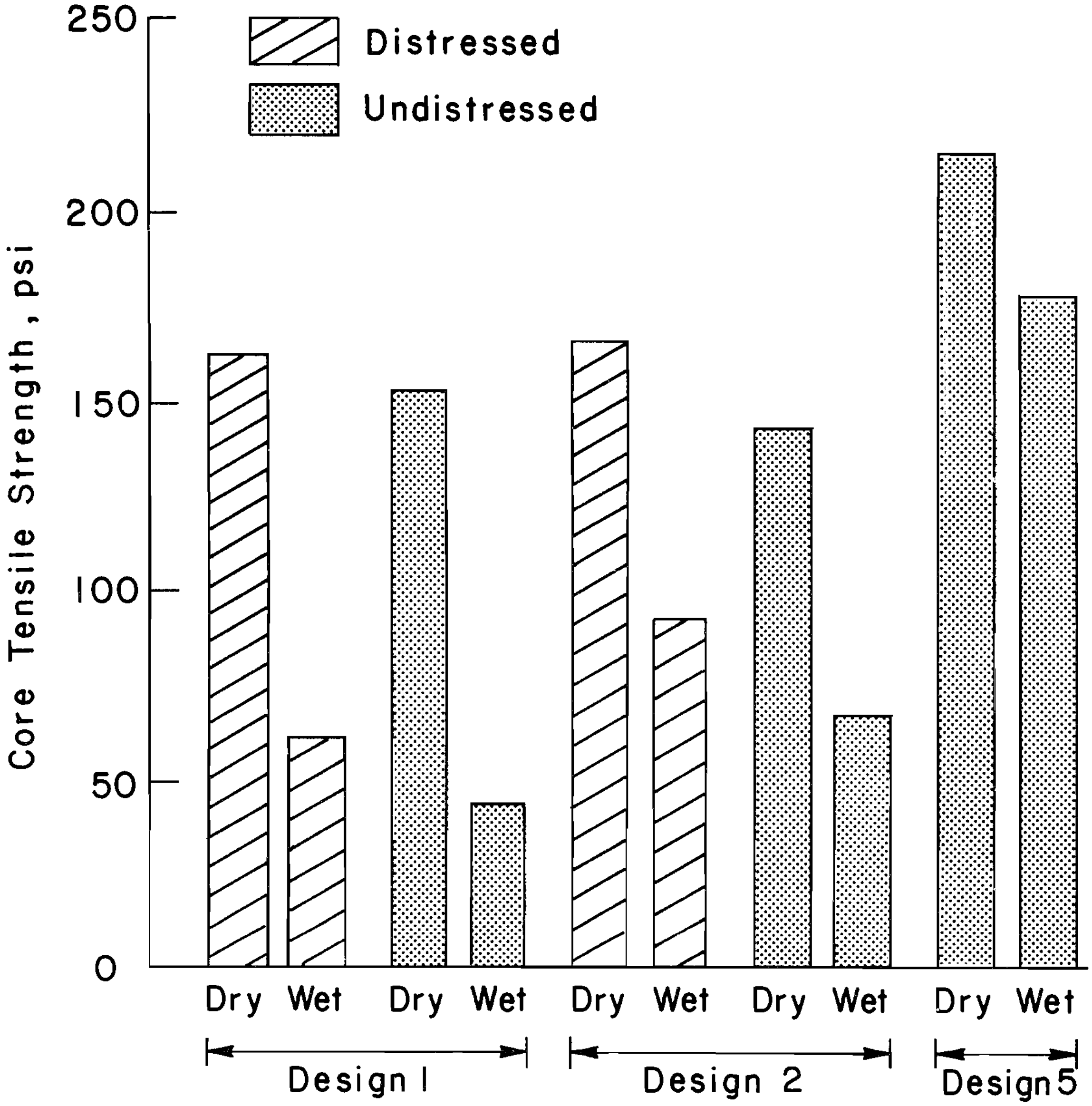

Fig 22. Average dry and wet core tensile strengths of the various mixture designs.

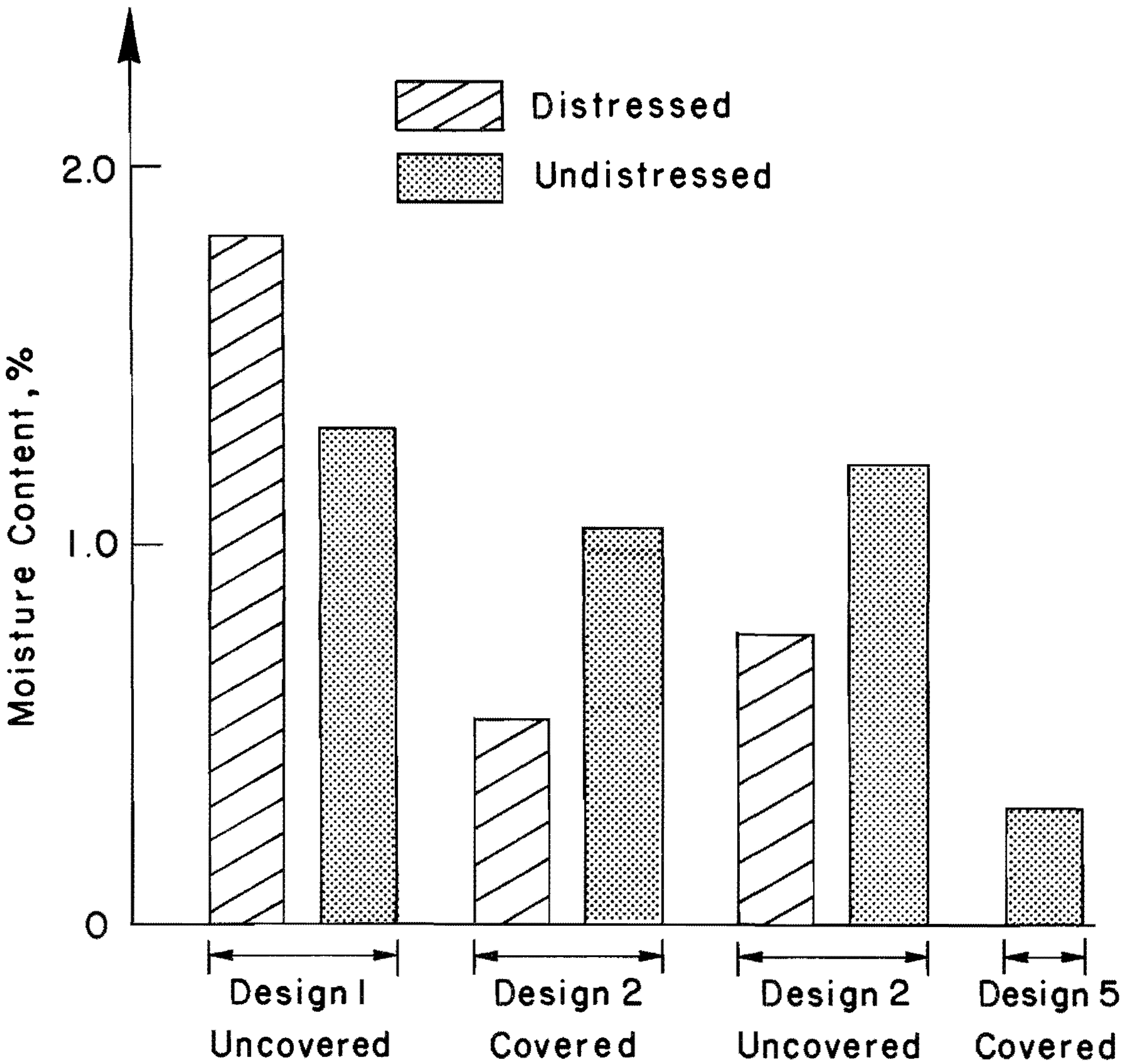

Fig 23. Moisture content of roadway slabs of the Type B base course.

MOISTURE CONTENTS

In-situ moisture content was determined for the Type B base course mixture from slab samples obtained at each coring location. The results summarized in Fig 23 indicate that the mixtures for designs 1 and 2 contained more moisture than for design 5. This is not unexpected since the Type B base course mixture from design 5 was covered by the Type D wearing course almost immediately after placement and because its density was significantly greater than the density of the other two designs. In addition, the areas of design 2 which were covered contained less moisture than design 2 areas which were not covered. Thus it appeared that moisture content was related to distress and needed to be evaluated.

MOISTURE SUSCEPTIBILITY

A series of tests was conducted to evaluate the moisture susceptibility of the various mixtures and involved Texas Freeze-Thaw Pedestal Tests and indirect tensile tests on moisture conditioned cores.

Texas Freeze-Thaw Pedestal Tests

Pedestal tests (Ref 5) were performed on both the individual and the combined aggregates of the Type B base course, with and without the antistripping agent. Previous work at The University of Texas at Austin has demonstrated that the test can distinguish between those Texas aggregates which are susceptible to stripping and those which are not. Aggregate-asphalt mixtures which fail in less than 10 to 15 cycles generally can be considered to be susceptible to moisture damage or stripping. The test procedure is contained in Ref 5 and is summarized in Appendix B. The results of the study are summarized in Fig 24. The results indicate that all three designs were susceptible to moisture damage and that all four individual sands were moisture susceptible, especially the Gem sand and the coarse sand. It can also be seen that there was no difference in the moisture susceptibility of the CCMC and Class Pit field sand and that the antistripping agent apparently was not effective.

Wet Tensile Properties

In addition to the dry tensile tests, the tensile properties were determined for cores at each location after subjecting each core to one of three

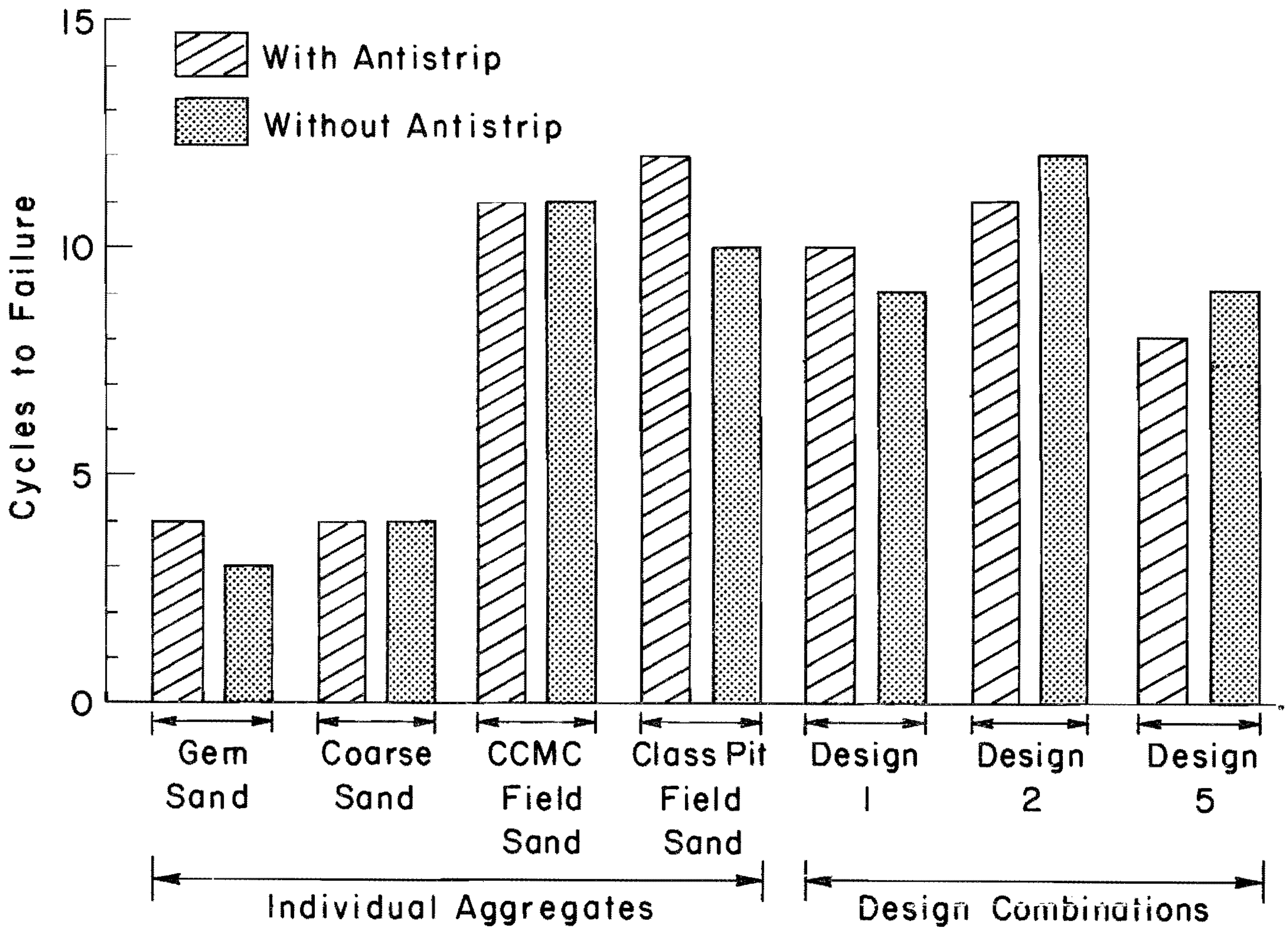

Fig 24. Texas freeze-thaw pedestal test results for individual aggregates and for Type B base course mixture.

moisture conditioning methods. The three methods of moisture conditioning, which are described in Appendix B, were

(1) vacuum saturated freeze-thaw,
(2) 7-day high vacuum saturation, and
(3) thermal cycling.

After conditioning, the tensile strength of each core was determined using the indirect tensile test. The results are contained in Fig 22.

Cores for design 5, after being moisture conditioned, retained a higher percentage of original dry tensile strength than those for either of the other mixtures. It has been suggested (Ref 6) that retention of less than about 70 percent of the dry tensile strength value when tested wet indicates a mixture that is moisture susceptible. When subjected to the most severe moisture conditioning methods, cores of mixtures from designs 1 and 2 generally retained less than 70 percent of the dry tensile strength, thus indicating their moisture susceptibility.

Laboratory specimens were prepared to evaluate the effect of density on the moisture susceptibility of the various mixtures (Appendix C). Specimens were compacted at a standard compactive effort and at an effort which produced densities equal to the field density of designs 2 and 5. The results are summarized in Fig 25.

The tensile strengths of the laboratory specimens of the design 2 mixture (Table C2) were similar to that of the cores; however, the tensile strengths for design 5 mixtures prepared at standard compaction effort show a significant loss (greater than 50 percent) when the mixture is subjected to moisture (Fig 25). These densities prepared with standard compaction effort were much lower than the field core densities which could not be attained in the laboratory (Table C2). This substantiates the findings of the pedestal tests and suggests that the retention of strength which occurred for the design 5 cores is due to the high densities achieved in the field which prevented or reduced moisture penetration. Thus, if moisture penetrates the design 5 mixtures, severe damage would be expected in this mixture as well as in the mixtures of designs 1 and 2.

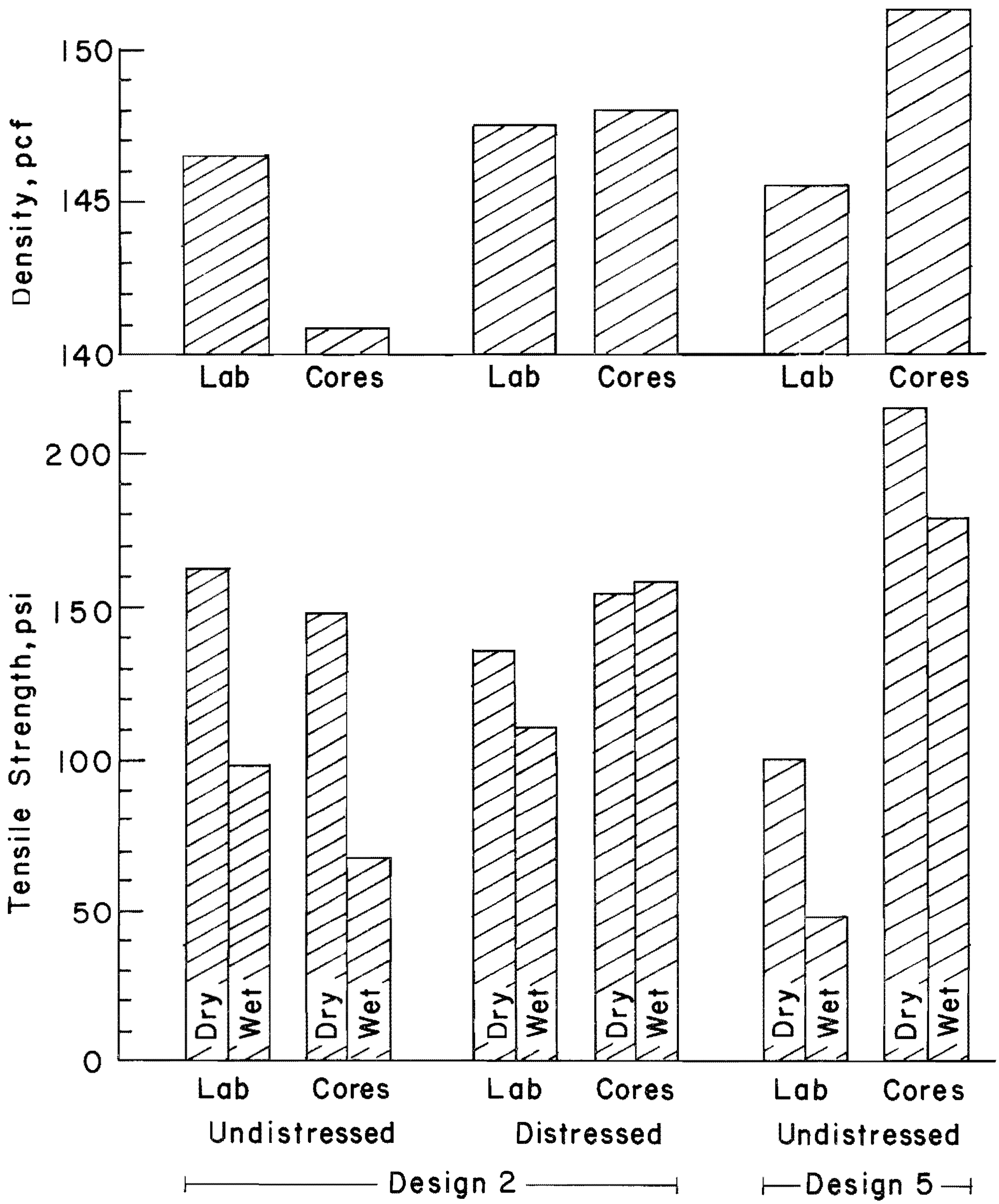

Fig 25. Effect of density on tensile strength after moisture conditioning.

CHAPTER 3. FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

The distress which occurred on IH-10 near Columbus, Texas, was in the form of rutting and shoving and was directly related to the Type B base course material. Major distress began in the design 1 material and was concentrated in designs 1 and 2. Essentially no distress occurred in design 5, although an extremely small amount of rutting had occurred at the time of the last condition survey. Designs 1 and 2 were similar with design 2 having only 0.2 percent more asphalt, while design 5 was markedly different, containing a different field sand and asphalt content.

CAUSES

The basic causes of the distress were:

(1) All aggregates and the resulting aggregate-asphalt combinations were highly susceptible to moisture damage.

(2) The antistripping agent was not effective in preventing moisture damage.

(3) In localized areas, the mixtures may have had higher asphalt contents which could have contributed to the failures.

RELATED FINDINGS

Other findings related to the distress and performance of the pavement and the various designs are as follows:

(1) Design 5 and the portion of design 2 placed after the winter had a lower field moisture content than the rest of the project. This was due to the following:

(a) Both sections had higher densities although the density of design 5 was substantially higher.

(b) Neither design section was exposed during the winter.

(c) Both design sections were covered soon after being placed in the spring.

(2) The sand equivalent values were more variable for designs 1 and 2; the sand source used for these designs contained localized areas of dirty sand. While this may have been a contributing factor, there is no evidence to indicate that this was a definite and direct cause of the severe distress in designs 1 and 2.

(3) The asphalt contents for the entire project were essentially within the specified tolerance of ±0.5 percent. The average asphalt content for mixture design 1, however, was slightly above the specified design asphalt content; design 2 was essentially equal to the design value; and design 5 was substantially below the design value. Designs 1 and 2 did have large percentages of asphalt contents above the design value, suggesting the possibility of localized areas which may have had slightly higher asphalt contents.

(4) The densities of design 5 and the portion of design 2 placed after the winter were significantly higher than for the rest of the project.

(5) The tensile strengths and the portion of the tensile strength retained after being subjected to moisture were much greater for design 5. This is attributed to the increased density which produced higher strengths and reduced moisture penetration.

The Hveem stabilities on the cores for design 5 were higher than for designs 1 and 2. There were essentially no differences between Hveem stabilities of laboratory prepared, job control specimens.

(6) The Texas freeze-thaw pedestal values and the retained tensile strength after moisture conditioning indicated that all aggregates are highly susceptible to moisture damage.

(7) All laboratory tests and field observations indicated that the anti-stripping agent was not effective in preventing moisture damage.

RECOMMENDATIONS

All information and findings indicate that construction was in compliance with specifications and the quality of construction was satisfactory. Nevertheless, it is felt that the following recommendations are justified.

(1) All asphalt-aggregate combinations including mixtures containing proposed antistripping agents should be tested to determine their susceptibility to stripping and moisture damage. Suggested test methods are the Texas Freeze-Thaw Pedestal Test (Ref 5), the Boiling Test (Ref 7), and the Wet-Dry Indirect Tensile Tests (Ref 8).

(2) Specified or required Hveem stability values should be increased to 35 to 40 for high volume highways.

(3) Specifications requiring a minimum level of compaction should be developed and definite density control requirements need to be implemented.

(4) Test procedures as used by D-9, the districts, and the residencies should be compared and checked periodically to insure that satisfactory and comparable results are being obtained.

(5) The specified tolerance of ±0.5 percent for asphalt content should be evaluated to determine whether tighter control is needed and is possible.

(6) The effect of aggregate gradation and the specified gradations should be evaluated to determine whether a tighter specification is required. Attention should be paid to the amount of material in the interval between sieves no. 40 and 80, the amount of fines, and the general shape of the gradation curve.

(7) Consideration should be given to establishing a construction sequence which minimizes exposure.

(8) Although not an apparent factor in this study, the viscosity of the asphalt should be evaluated to determine the long-term effects on stability and strength. Special emphasis needs to be placed on the influence of plant temperatures, especially with respect to drum dryers which are more variable and operate at lower temperatures.

(9) Design and job control testing needs to be increased to insure adequate design and construction. In addition, consideration should be given to implementing and, if necessary, developing new tests which could provide more meaningful information with respect to material and pavement performance. This represents a very small expenditure of funds in comparison to the overall cost of a project and, more importantly, to the cost of the consequences to the short-term and long-term performance of the pavement.

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APPENDIX A

MIXTURE DESIGN INFORMATION

APPENDIX A. MIXTURE DESIGN INFORMATION

All asphalt concrete mixtures were produced according to Texas Standard Specifications (1972) under Item 340, "Hot Mix Asphaltic Concrete Pavement" (Ref 9). Design was based upon a minimum modified Hveem stability value of 30 at a density of 97 percent of the maximum theoretical density as computed by the method outlined in Ref 9.

An AC-20 produced by Exxon in Baytown, Texas, was used in the mix; and a liquid anti-strip agent NEA-3, produced by Norwood Engineering Associates, was added to the asphalt on the job at the rate of 20 pounds per ton of asphalt. This agent had been used on several other overlay projects throughout the state and had been approved by the Materials and Test Division, D-9, Texas DHT.

Several sources of aggregates were used for the Type D level-up, Type B base course and Type D wearing course mixtures. Except for the coarse aggregate in the Type D wearing course and the two field sands, all of the aggregates used were siliceous river deposits and came from the Dvorak pit near Columbus. A crushed sandstone from the Wagner pit near Moulton, Texas, was used in the Type D wearing course. The first of two siliceous field sands was produced by Columbus Construction Materials Company (CCMC) and the second by the contractor from the Class pit near Columbus.

TYPE D LEVEL-UP

The Type D level-up consisted of a blend of

(1) gem sand,
(2) coarse sand, and
(3) CCMC field sand.

The design asphalt content varied from 5.3 to 5.5 percent. Figure A.1 shows the location where the Type D level-up was placed.

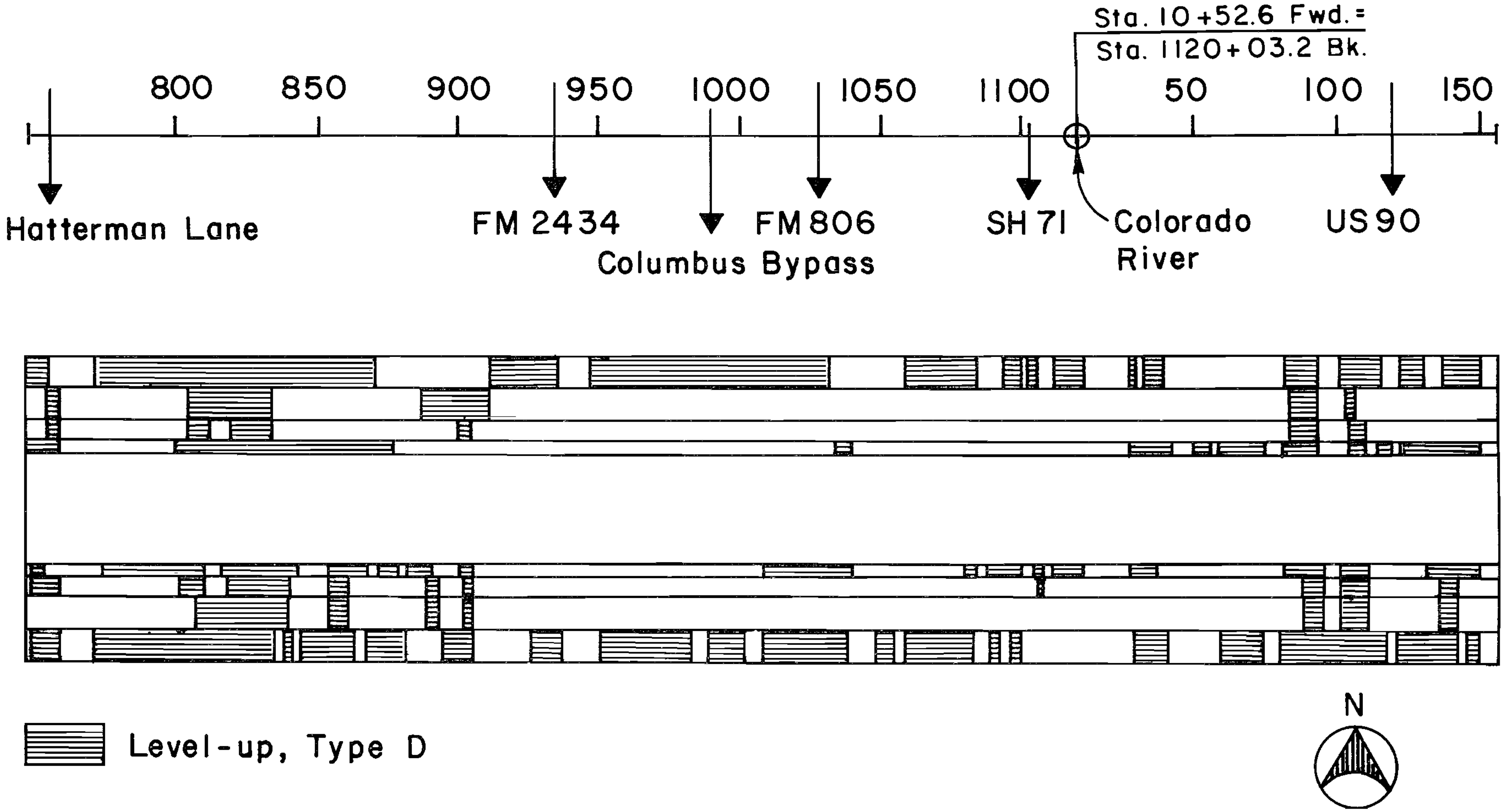

Fig A.1. Location of Type D Level-up.

TYPE B BASE COURSE

The Type B mixture consisted of a blend of

(1) coarse aggregate,
(2) gem sand,
(3) coarse sand, and
(4) field sand from either of the two previously mentioned sources.

Three different mix designs, Designs 1, 2, and 5, were developed and used. Table A.1 summarizes the three Type B mixture designs and Fig 9 shows the location where each design was placed. The CCMC field sand, which was used on approximately 75 percent of the project (designs 1 and 2), only marginally met sand equivalent requirements and was quite variable with pockets of dirty material. Because of these pockets the Class field sand was used (design 5) which also barely met sand equivalent requirements but seemed to be more uniform. The design asphalt contents were 4.7 and 4.9 percent for designs 1 and 2, respectively.

TYPE D WEARING COURSE

The Type D wearing course was a mixture of

(1) crushed sandstone,
(2) gem sand,
(3) coarse sand, and
(4) Class pit field sand.

The design asphalt content varied from 6.2 to 6.5 percent by weight of the total mixture.

TABLE A.1. MIXTURE DESIGN FOR TYPE B BASE COURSE

Sieve Interval	Percent Retained: Specifications	Design 1*	Design 2**	Design 5***
7/8"- 3/8"	20 - 50	34.6	37.6	34.5
3/8"- #4	10 - 40	16.0	14.6	14.9
#4 - #10	5 - 25	10.9	10.5	11.5
+ #10	55 - 70	61.5	62.7	60.9
#10 - #40	0 - 30	11.9	12.1	14.9
#40 - #80	4 - 20	12.0	9.9	12.7
#80 - #200	3 - 20	8.4	8.0	5.2
- #200	0 - 6	1.5	2.4	2.3
Asphalt	3.5 - 7.0	4.7	4.9	4.0

*Design 1

40% Coarse Aggr.
27% Gem Sand
13% Coarse Sand
20% CCMC Pit Field Sand

**Design 2

37% Coarse Aggr.
26% Gem Sand
11% Coarse Sand
26% CCMC Pit Field Sand

***Design 5

37% Coarse Aggr.
26% Gem Sand
11% Coarse Sand
26% CCMC Pit Field Sand

APPENDIX B

DESCRIPTION OF TESTING PROGRAM

AND PROCEDURES

APPENDIX B. DESCRIPTION OF TESTING PROGRAM AND PROCEDURES

TESTING

The testing consisted of four basic testing programs. First, compliance tests were performed to determine if in-place properties satisfied DHT specifications. Second, engineering properties were estimated to determine whether these problems related to or could be used to explain observed field performance. Third, water damage tests were performed to determine the moisture susceptibility of the mixtures. Fourth, test specimens were molded in the laboratory using virgin materials, field temperatures, and field densities to determine the effect of these parameters on tensile strength and Hveem stability of prepared laboratory specimens.

Compliance Tests

Absolute density was determined for each core in accordance with test method Tex-207-F (Ref 9). Three cores from each sampling location were tested to determine Hveem stabilities, according to test method Tex-208-F (Ref 9).

Engineering Properties

Dry and wet tensile strengths and static and resilient moduli of elasticity were determined for several cores at each location using either the static or repeated-load indirect tensile test method as developed by Kennedy (Refs 10 and 11). For the wet testing, three methods of moisture conditioning were used as follows.

(1) Vacuum saturated freeze-thaw (VSFT) - involved soaking specimens for 30 minutes under a 15-inch mercury (Hg) vacuum and soaking for an additional 30 minutes at atmospheric pressure at 75°F, freezing 15 hours at 0°F, soaking 24 hours in a water bath at 140°F, and finally soaking 3 hours in a water bath at 75°F before testing.

(2) Seven day high vacuum saturation - involved soaking specimens for seven days at room temperature, 75°F, under a 15-inch mercury vacuum. After the vacuum saturation period, the specimens were immediately tested.

(3) Thermal cycling (TC) - involved soaking specimens for 30 minutes under a 15-inch mercury vacuum and soaking for an additional 30 minutes at atmospheric pressure at 75°F, followed by 18 cycles of freezing and thawing. One freeze-thaw cycle consists of four hours at 10°F followed by four hours at 120°F. Every other cycle was followed by eight hours at 75°F. The specimen was placed in a 75°F water bath during the last eight hours rest period. Specimens were then immediately tested.

Texas Freeze-Thaw Pedestal Test

In addition to the moisture conditioning tests on cores and laboratory prepared specimens, the Texas Freeze-Thaw Pedestal Test was performed to evaluate the moisture susceptibility of the individual aggregates or combination of aggregates used on the project.

The pedestal test was developed and evaluated by the Center for Transportation Research (Ref 5) based on a procedure proposed by the Laramie Energy Technology Center (Ref 12), as a laboratory technique for evaluating the moisture susceptibility of asphalt mixtures. Small briquets approximately 0.75 in. high by 1.375 in. diameter are molded. The briquets are composed of a uniform-sized mixture of aggregate or combinations of aggregates mixed with the asphalt and any additive, such as anti-strip agents, that are to be evaluated. The one-sized material ensures easy penetration of water and minimizes particle interlock so that the briquet properties are largely dependent on the properties of the asphalt-aggregate bond. After molding, the briquets are placed on a pedestal and submerged in individual jars containing distilled water. The specimens are then subjected to alternating freeze-thaw cycles and the number of cycles required to cause the briquet to fail is determined. This test not only gives an indication of the susceptibility of an aggregate to moisture, but can also be used to evaluate the effectiveness of various anti-strip agents to enhance the bond between asphalt and aggregate.

Work by Kennedy et al (Ref 5) has suggested that materials which fail in less than 10 to 20 cycles are generally considered to be moisture susceptible. Materials which sustain more than twenty-five cycles are currently considered not to be susceptible to water damage.

APPENDIX C

LABORATORY COMPACTION STUDY

APPENDIX C. LABORATORY COMPACTION STUDY

A laboratory compaction study was initiated in order to evaluate the effect of actual in-place field road density on moisture susceptibility and Hveem stability. Specimens were compacted to simulate field conditions of gradation, asphalt content, compaction temperature and density.

Three sites were selected which had job control records available concerning gradation, asphalt content, and compaction temperature. In-place density information was available from core data.

Two of the sites were in design 2 on either side of a construction joint located at approximately station 950 eastbound, outside lane, and identified in the laboratory study as group 2U (undistressed) and groupd 2D (distressed). The corresponding coring locations for the two sites were 25 and 26, respectively. The third site was located in design 5 at station 149+50 westbound, outside lane, coring location 21, and was identified as group 5U (undistressed).

A fourth group of laboratory specimens was compacted and identified as 5US (undistressed, slab). This group varied from the 5U group in that the gradation and asphalt content used was based on extraction results from a slab cut from the roadway and was of a finer gradation and contained a higher percentage of asphalt. A summary of the laboratory study compaction conditions is shown in Table C.1.

For each group nine specimens were prepared using standard DHT laboratory procedures and nine using field compaction temperature with the compactive effort being varied to obtain field core density. For each group of nine specimens three each were tested to determine dry tensile strength, wet tensile strength, and Hveem stability. The wet tensile strengths were determined after subjecting the specimens to the thermal cycling (TC) method of moisture conditioning.

Dry and wet tensile strengths of the laboratory compacted specimens are shown in Table C.2 along with the core values from each of the locations that

TABLE C.1. SUMMARY OF LABORATORY STUDY COMPACTION CONDITIONS

Compaction Group	Source of Gradation and Asphalt Content	Asphalt Content, %	Compaction Temperature, °F	Compactive Effort	Density, pcf	
					Core	Achieved
Design 2 Undistressed (2U)	Job Control Records	4.4	250 (DHT Standard)	DHT Standard	N/A	144.9
			290*	Variable	146.6	146.5
Design 2 Distressed (2D)	Job Control Records	4.9	250 (DHT Standard)	DHT Standard	N/A	148.2
			300*	Variable	146.7	147.5
Design 5 Undistressed (5U)	Job Control Records	3.5	250 (DHT Standard)	DHT Standard	N/A	145.6
			275*	Variable	151.5	148.0
Design 5 Undistressed, slab (5US)	Roadway Slab Extraction	4.1	250 (DHT Standard)	DHT Standard	N/A	143.8
			275*	Variable	151.5	—

*Compaction temperature from job control records

TABLE C.2. DRY AND WET TENSILE STRENGTHS OF LABORATORY COMPACTED SPECIMENS COMPOSED OF VIRGIN MATERIALS COMPARED WITH VALUES FROM CORRESPONDING CORE LOCATIONS

Compaction Group	Compaction Criteria	Dry Tensile Strength				Wet Tensile Strength*			
		Compacted		Cores**		Compacted		Cores**	
		Mean, psi	Number Tested	Mean, psi	Number Tested	Mean, psi	Number Tested	Mean, psi	Number Tested
Design 2 Undistressed *** (2U)	DHT Standard	129	3	149	3	55	3	69	3
	Field Core Density	162	3			96	3		
Design 2 Distressed (2D)	DHT Standard	137	3	156	3	152	3	158	3
	Field Core Density	138	3			110	3		
Design 5 Undistressed (5U)	DHT Standard	100	3	219	3	47	3	188	2
	Field Core Density	-	-			-	-		
Design 5 Undistressed, slab (5US)	DHT Standard	123	3			30	3		
	Field Core Density	-	-			-	-		

*Wet tests performed using thermal cycling conditioning described in Appendix B.

**Cores from locations 21, 25, and 26.

***Moderate distress at time of final survey.

the compacted specimens simulated. Hveem stability values for each group along with the corresponding core values are shown in Table C.3.

TABLE C.3. HVEEM STABILITY OF LABORATORY COMPACTED SPECIMENS COMPOSED OF VIRGIN MATERIALS COMPARED WITH VALUES FROM CORRESPONDING CORE LOCATIONS

Compaction Group*	Compaction Criteria	Hveem Stability			
		Lab. Prepared		Cores	
		Mean	No. Tested	Mean	No. Tested
Design 2 Undistressed (2U)	DHT Standard	42	3	18	3
	Field Core Density	39	3		
Design 2 Distressed (2D)	DHT Standard	34	3	10	3
	Field Core Density	36	3		
Design 5 Undistressed (5U)	DHT Standard	42	3	38	3
	Field Core Density	-**	3		
Design 5 Undistressed, slab (5US)	DHT Standard	39	3		
	Field Core Density	-**	3		

*Designation contains two elements - the number is for the mixture design, and the letter represents a field condition:

U - undistressed location
D - distressed location
US - slab sample from undistressed location

**Could not achieve target density with gyratory shear compactor