A STUDY OF TRANSVERSE CRACKING OF ASPHALT PAVEMENTS IN MANITOBA

A Thesis

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Master of Science

by

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SUMMARY

Transverse cracking of bituminous pavements is a form of pavement distress which induces losses in pavement performance and reduction in pavement service life. To alleviate the problem, more information is needed about the mechanism of transverse pavement cracking and the low temperature properties of highway construction materials.

The Ste. Anne Test Road was constructed in Manitoba in 1967 and was designed for the study of transverse pavement cracking. Twenty-nine test sections were constructed incorporating a number of different materials and pavement structures, judged to be potentially important in the study of transverse cracking. The test road was instrumented to measure and record the thermal regime of the pavements at regular time intervals. Several sections were instrumented to detect the initiation of transverse cracking. The transverse cracking performance of the pavements, based on two years of observations, is presented in the thesis.

A study of the mechanics of the fracture phenomenon indicated that most of the transverse cracking initiated during a prolonged low temperature cycle which has an average recurrence interval in southern Manitoba between five and ten years. Based on observations, crack detection instrumentation data and an x-ray study, transverse cracking was judged to initiate at the pavement surface.

Analyses of the effect of the variables on the transverse cracking frequency of asphaltic concrete pavements indicated that transverse cracking can be alleviated through the selection of asphalts of the proper type and grade. With respect to the effect on the frequency of transverse pavement cracking, it was found that:

1. Asphalt type and grade were the dominant variables,
2. Asphalt content of the mix was not significant,
3. The addition of a small amount of portland cement filler was not significant,
4. Traffic loading was not significant,
5. There was an interaction between the asphaltic concrete and the subgrade at low temperatures,
6. The frequency of transverse cracking varies inversely as the thickness of the asphaltic concrete.

ACKNOWLEDGMENTS

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CHAPTER I

INTRODUCTION

Transverse cracking is a form of asphalt pavement distress which results in the fracture of the pavement surface perpendicular to the direction of vehicular travel. These cracks generally appear at regularly spaced intervals. The problem is indigenous to the low temperature environmental regions of Canada and the northern United States. A pavement exhibiting regular transverse cracking is shown in Figure 1.

Figure 1. Bituminous Pavement Exhibiting Transverse Cracking at Regular Intervals.

Transverse cracking poses a great problem in the prairie provinces where severe cracking of pavements has been reported. Studies in Alberta (1)* have revealed transverse crack frequencies higher than 100 per mile and up to 450 per mile while in Saskatchewan (2) the average has been reported to be 100 transverse cracks per mile. A 1964 survey in Manitoba of 312 miles of flexible pavements of various ages and structures revealed an average of 211 transverse cracks per mile. The distribution of the intervals between cracks or "slab" length is illustrated in Figure 2.

The Ste. Anne Test Road, the details of which are given in Chapter III, was designed to provide a basis for the investigation of transverse pavement cracking and was constructed in 1967.

Scope of the Investigation

The objectives of the investigation reported in this thesis are:

1. To indicate the prevalence of transverse pavement cracking and to describe the effects of transverse cracking on pavement performance,
2. To describe the design and provide an evaluation of the quality of the materials incorporated in the Ste. Anne Test Road,
3. To evaluate the performance of the pavements at

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*Numbers in parentheses refer to REFERENCES at end of thesis.

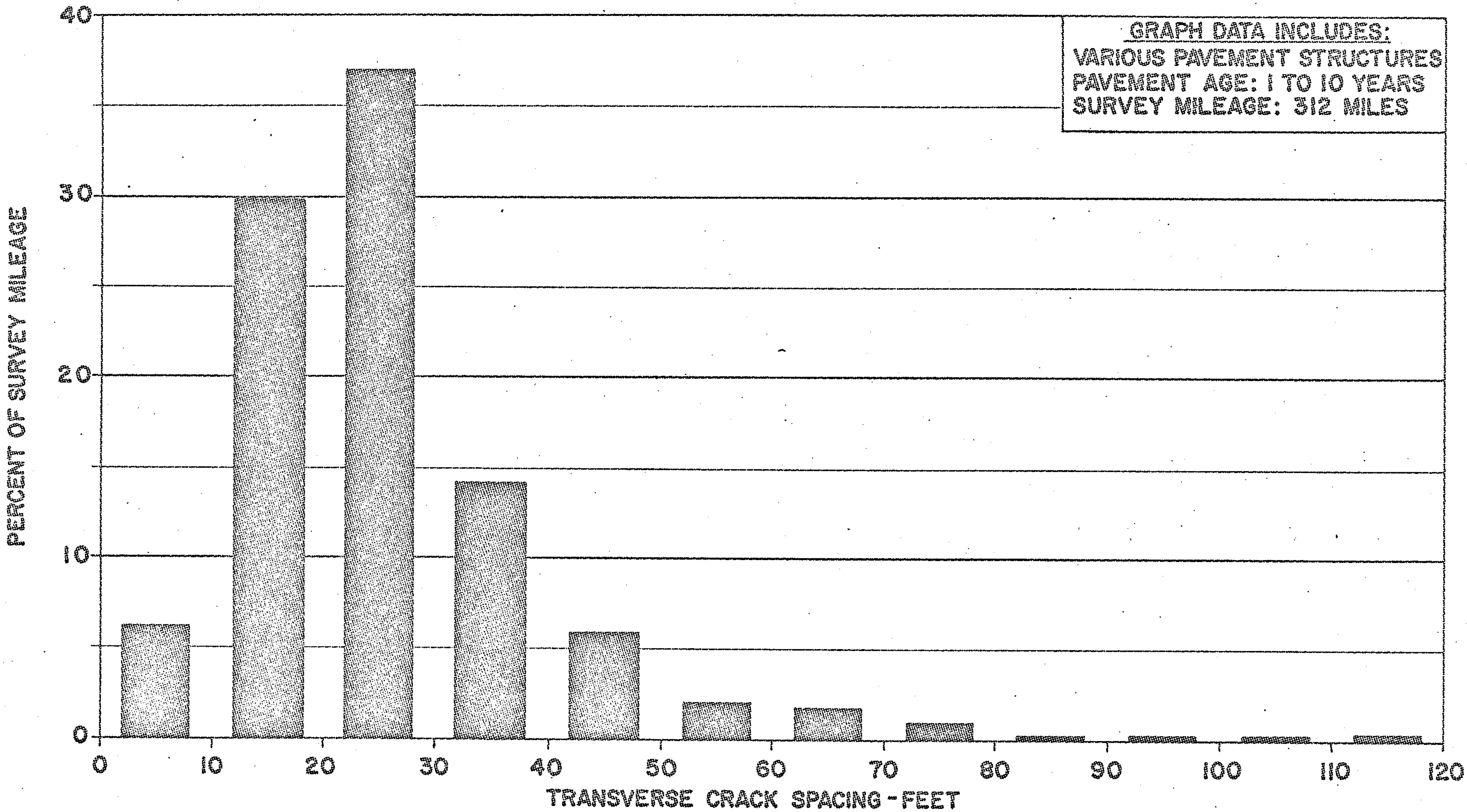


Figure 2. Transverse Cracking Frequency of 150-200 Penetration Asphalt Pavements with A-6 and A-7-6 Clay Subgrades.

low temperatures and to isolate the variables which have a bearing on the transverse cracking frequency of pavements,

4. To establish the temperature conditions at the initiation of transverse cracking,

5. To investigate the mechanism of transverse crack propagation.

Organization of the Thesis

The thesis is organized to describe available information regarding transverse pavement cracking, to describe the Ste. Anne Test Road and to provide an analysis of the principal findings to date, two years after the construction of the test road. Available information on transverse pavement cracking based on literature research and past observations in Manitoba is cited in Chapter II. The design of the Ste. Anne Test Road and the quality of the materials are described in Chapter III. The transverse cracking of the test road pavements observed to date is reported in Chapter IV. The temperature conditions at initiation of transverse cracking and observations regarding the mechanism of transverse cracking are described in Chapter V. The principal findings of the investigation are summarized in Chapter VI with suggestions for future research.

CHAPTER II

TRANSVERSE CRACKING INFORMATION SURVEY

Consequences of Transverse Cracking

For a period after the occurrence of transverse cracking, the riding quality of the pavement is not affected by the presence of the transverse cracks. However, Benkelman beam rebound measurements indicate that the pavement in the region of the crack is subject to stress concentrations from traffic loading. Faulting of the pavement surface at the cracks has been reported in a few instances. Depending upon the width, the pavement crack provides a condition for the ingress of debris and surface moisture into the underlying materials. If the subgrade soil is swell susceptible, heaving of the pavement may result adjacent to the crack forming a ridge across the pavement. In the case of a sand subgrade, subsidence of the pavement surface may result, forming a depression across the pavement at the transverse crack.

In 1962, Wicks (3) assessed the deformation of pavement surfaces in Manitoba arising from transverse cracking. Using a ten-foot beam as a basis of reference, he measured the vertical displacement of the pavement at transverse cracks. The survey results of 460 miles of transversely cracked pavements of the 2,164 mile bituminous pavement highway system are presented in Table I. The data indicated that 22 percent of the pavements exhibited ridges in excess of 0.3 inch in height at transverse cracks.

Table I. Vertical Displacement of Pavement at Transverse Cracks

Height (Inches)	Percent of Survey Mileage
No Displacement	45
0.0 to 0.3	33
0.3 to 0.6	13
0.6 to 1.0	6
1.0 +	3

The reduction in pavement riding quality, resulting from transverse cracking, has resulted in a noticeable decrease in the present performance rating of some asphalt pavements. Shields (4) has reported a significant increase in roughness index as a consequence of transverse cracking, resulting in a reduction in the present performance rating. Anderson et al (5) describe cases where severe performance losses have resulted from ridging of pavements with swelling type subgrades. He cited an example where a pavement with a projected life of approximately 12 to 16 years achieved its terminal serviceability level after only four years.

Maintenance measures have proved to be costly and unsatisfactory in alleviating the effects of transverse pavement cracking. Filling the cracks with asphalt appears to reduce the effective crack width rather than providing an impervious seal. Winnitoy (6) has approximated the cost to be $200 per mile annually in Saskatchewan where the average transverse crack frequency is 100 per mile. In Manitoba, this cost ranges from $15 to $400 per mile. Heater planing of the ridges has proven to be an expensive temporary measure, ineffective in preventing further progressive vertical development of the ridges. The average cost of heater planing the ridges on a nine mile highway section in Manitoba was $600 per mile. This did not include the cost of resealing the transverse cracks. The filling of the depressions at the transverse cracks with bituminous mix has been unsatisfactory in many cases in restoring the riding quality of the pavement.

The overall effect of transverse pavement cracking is reflected by the decrease in the present serviceability index and a reduction in the performance life. The economic losses, incurred through the increase in pavement maintenance costs and the reduction in pavement life, may only be assessed qualitatively and described as ranging from moderate, in instances where little ill effects have resulted from pavement cracking, to great, as described in several cases previously.

The Mechanism of Transverse Pavement Cracking

On the basis of observations by the Manitoba Department of Transportation, transverse cracking of asphaltic concrete does not appear more frequent on heavily travelled highways than on low traffic volume roads. This has led to the belief that transverse cracking is non-load associative, with traffic playing only a minor role if any at all.

Transverse cracking of the asphaltic concrete surface has been attributed to the development of tensile stresses originating either externally, from the subgrade or base, or internally, from within the asphaltic concrete, or due to a combination of the above. Some contributing causes have been ascribed to contraction through temperature change or shrinkage through volume change of the pavement materials and to bending of the asphaltic concrete coupled with the contraction resulting from thermal stresses. The exact mode of initiation or progression of transverse cracking has not been defined.

In the prairie provinces, transverse pavement cracking has been observed to occur sometime after the inception of cold weather. From observations it appears that thermal contraction is the major causative factor. Neither the thermal regime of the pavement structure nor the exact nature of the interaction between the asphaltic concrete surface and its sub-layers at the time of cracking has been established. Regardless of where the excessive forces originate, cracking is manifested in the asphaltic concrete surface.

Two main hypotheses exist regarding the mechanism of inception and propagation of low temperature cracking. According to one mechanism, transverse cracking is the result of a prolonged cold spell followed by a sudden rise in temperature. The warming trend creates a decrease in the strength of the asphaltic concrete surface and results in pavement cracking due to excessively high thermally induced tensile stresses still present in the lower portion of the asphaltic concrete (7,1,2). According to another theory, cracking may ensue as a result of a rapid temperature drop to an extremely low level (8,9,10) or from a prolonged cold temperature period (7). This may result either in instantaneous cracking of the asphaltic concrete or in the development of a microcrack at the pavement surface, where the thermal stresses are in excess of the tensile strength. As the cross-sectional area of the asphaltic concrete is reduced, the microcrack may propagate downward through the asphaltic concrete layer.

Prior to the observations of transverse cracking in the Ste. Anne Test Road pavements, observations of 150-200 penetration grade asphalt highway pavements in Manitoba had indicated that most of the cracking occurred during the first two winters after the construction with little or no transverse cracking thereafter. The cracking appeared to initiate during January or February, the cold weather months, giving credence to the latter type of cracking mechanism described above. However, pavements in other environmental regions may be subject to different transverse cracking mechanisms. Such is the case in Alberta where a pavement was observed to crack audibly during a period of sudden increase in air temperature (11).

Factors Influencing Transverse Pavement Cracking

It is conceivable that the mechanism and the frequency of transverse pavement cracking in a given climatic region could be dependent upon the design of the pavement structure and the properties of the materials under the prevailing temperature conditions. While the pavement cracking mechanism may not be completely understood, various investigators have isolated a number of pavement design and materials variables which are believed to have bearing on the frequency of transverse cracking.

Observations of in-service pavements have implicated the use of certain grades and types of asphalts. Marker (12) cited that " . . . lower penetrations (of recovered asphalt) were associated with the more severely cracked projects (in Nevada)." This has also been substantiated by observations in Manitoba (13,14,15). Anderson et al (5) have concluded that " . . . evidence from field observations suggests a cracking preponderance associated with certain asphalt sources (in Alberta)." Culley (2) and McLeod (15) report that in Saskatchewan reduced transverse cracking has been observed in higher viscosity type and softer penetration grade asphalt pavements than in lower viscosity type and harder penetration grade asphaltic pavements respectively. Also, age hardening of asphalts in bituminous mixes is believed to be a factor.

Other variables which have been isolated as being suspect to contributing to pavement cracking include absorptive aggregates (16,17), low asphalt content, and large residual stresses caused by considerable difference in the coefficients of expansion of mineral aggregates and asphalt (18). Laboratory studies have also indicated that the tensile strength of asphaltic concrete may be increased by the addition of mineral filler if the filler-bitumen ratio is optimized (19).

Pavement structures of a certain design may indicate greater susceptibility to transverse pavement cracking than others. Fromm (20) has suggested that the strength of asphaltic concrete pavement could be a factor on the basis of observations of a pavement section in Ontario where a 3½-inch asphaltic concrete surface exhibited a lower transverse crack frequency than an adjoining section with a two-inch layer.

While much emphasis has been placed on the asphaltic concrete components, indications are that the volume change of the underlying materials plays a very important role in transverse pavement cracking. Hamilton (21) has found that certain partially saturated clay specimens, when subjected to uniaxial freezing, may contract more than six percent. Figure 3 illustrates transverse cracking of a gravel surfaced road in Manitoba with a heavy clay subgrade which has cracked as a result of thermal effects.

Figure 3. Transverse Cracking of a Gravel Road with a Heavy Clay Subgrade

Figure 4 shows a transverse crack running across the gravel shoulder and part-way into the pavement. However, this does not implicate the subgrade as being entirely responsible for transverse cracking but demonstrates that it is another factor that must be considered in contributing to the transverse cracking of asphalt pavements.

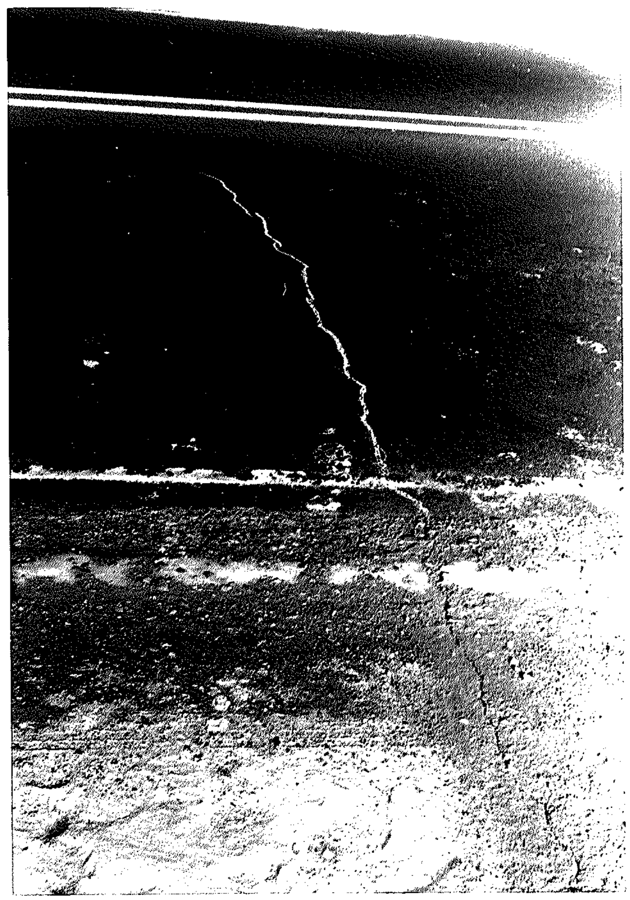


Figure 4. Transverse Crack in a Gravel Shoulder and Part-way Across the Pavement

Figure 5 illustrates a "dummy" contraction joint sawed part-way into the asphalt concrete surface of a pavement in Manitoba in a research project designed to investigate the feasibility of controlling transverse pavement cracking. The photograph demonstrates that restraining forces at the asphaltic concrete-gravel base course interface have caused transverse cracking to propagate into the gravel shoulder as a consequence of the contraction of the asphaltic concrete surface.

Figure 5. Cracking of Gravel Shoulder at a Pavement Contraction Joint

Radical differences in transverse crack frequency within certain projects have indicated that non-uniformity of materials and construction control measures may have a bearing on thermal pavement cracking. Frequent cracking at construction joints and through weakened sections of pavements in Manitoba partially support this viewpoint. For a number of years it has been observed that new pavements, which have been cored to determine asphaltic concrete thickness and density, later exhibited transverse pavement cracking with the crack passing through the center of the core hole.

Figure 6 illustrates the case where the location of a four-inch diameter core hole, contributing to a reduction in cross-sectional area of only 0.14 percent of the four-inch thick asphaltic concrete, was instrumental in predetermining the location of the transverse crack.

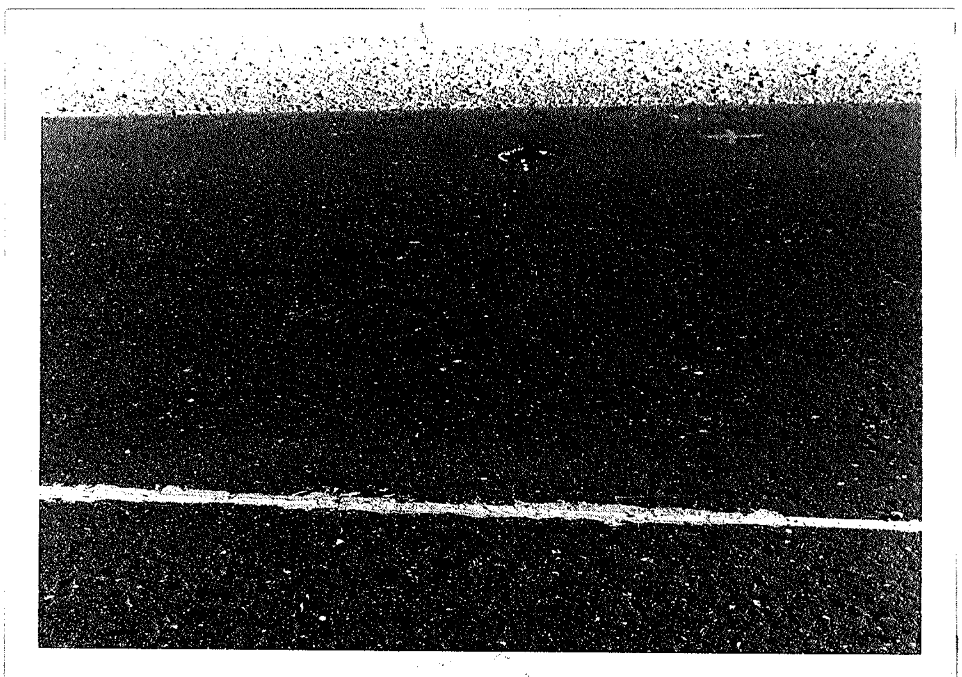


Figure 6. Transverse Pavement Crack Initiated by Pavement Coring

The various factors described appear to indicate that in some instances transverse cracking may be the result of one or more causative agents. Although results are not yet available from full-scale experiments to establish the exact mechanism of transverse pavement cracking, field observations and comparisons have served to isolate certain variables which have a bearing on this problem. These observations have implied that it is necessary to be more selective in the use of materials and pavement designs with due consideration being given to their performance under the prevailing climatic conditions.

Design Methods Incorporating Environmental Factors

The transverse cracking of asphalt concrete is only one type of distress mechanism that must be considered in a design system for asphalt concrete pavements. Epps (22) states that "Since a pavement is a complex structure which is subject to various loading conditions and since it must perform under a variety of environments, the use of systems engineering provides a means of organizing the various segments or components of the total problem into an understandable framework." This concept has also been described by Hutchinson and Haas (23) and Hudson et al (24). The design system advanced in the latter reference consists of:

1. inputs to the system comprising action and interaction of load, environmental, construction and maintenance variables,

2. physical dimensions of the pavement structure and the basic properties which characterize material behavior,
3. outputs consisting of primary response, described by the behavior of the materials, and limiting response, defined by distress modes,
4. decision criteria based on economic and performance factors,
5. feedback and interaction among the different components of the total system.

Within such a systems engineering framework, the properties of the materials are considered a major design element. For asphaltic concrete surface courses the following mix properties must be considered in mix design (22):

1. stability,
2. durability,
3. flexibility,
4. fatigue resistance,
5. skid resistance,
6. permeability or imperviousness,
7. fracture strength,
8. workability,
9. thermal characteristics.

To optimize pavement performance, a balanced design is necessary with respect to the above asphalt concrete properties. The response of asphaltic concrete to low temperatures is only one design consideration among many and care must be taken since a design imbalance in favor of low temperatures could result in the development of other undesirable performance characteristics.

As low temperature is a factor in the transverse cracking of asphalt pavements, design approaches to counter this effect should be on a regional basis and should include the probability of recurrence of low temperature levels. It is conceivable that the physical properties of an asphaltic concrete or its components could be assessed for a predetermined low temperature level and its suitability for use then based on a limiting physical parameter or parameters relating to stresses and deformations.

For the asphalt fraction of the mix, Van der Poel (25) has formulated a system for determining the stiffness of any bitumen at any temperature and time of loading. Heukelom and Klomp (26) have related stiffness modulus of the binder to the stiffness modulus of the mix and Heukelom (27) has extended the concept of the stiffness modulus of the bitumen to the tensile strength of the mix. Using a thermal contraction coefficient, Hills and Brien (9) have computed the induced thermal stresses at various temperatures in a restrained asphaltic concrete specimen and with the aid of Heukelom's mix tensile strength data have estimated fracture of the specimen to occur at a temperature when the thermal stress exceeds the tensile strength. Thus, a method for estimating asphaltic concrete fracture temperature exists but still requires validation for in-service pavements.

Another method which may be acceptable for assessing the suitability of a given bituminous mix to resist transverse cracking in a specific climatic environment may be through the use of a standardized index test. The tensile splitting test of cylindrical asphalt concrete specimens is described by Christison (28) Breen and Stephens (29), Anderson and Hahn, (30) and Haas and Anderson (31). The usefulness of such a test would be dependent upon the degree of correlation of the test results with observed field performance of pavements.

CHAPTER III

THE DESIGN AND QUALITY CONTROL OF THE STE. ANNE TEST ROAD

Characteristics of the Test Road

The Ste. Anne Test Road is a joint project of the Manitoba Highways Department and Shell Canada Limited designed to study the transverse cracking of asphalt pavements. A number of materials and pavement structures have been incorporated in test sections. The experiment has been designed to yield insight into the mechanism of transverse crack initiation and propagation, and to isolate the significant variables which may have a bearing on the frequency of transverse cracking under the climatic conditions prevailing in southern Manitoba.

The test road is situated approximately 25 miles east of Winnipeg in the vicinity of Ste. Anne, Manitoba. This location was selected as it permitted construction of one-half of the road on a heavy clay subgrade, the bed of the prehistoric glacial Lake Agassiz, and the other half on a sand subgrade, the old sandy beaches of Lake Agassiz.

The ambient temperatures in the region range from 100° F to -45° F and the freezing index is generally between

3,000 to 4,000 degree days of frost. The annual precipitation is approximately 20 inches.

The test road was constructed in 1967 and is a newly constructed section of highway. It constitutes the two west bound lanes of the four-lane divided Trans-Canada Highway and carries an average of 1,250 vehicles per day. The truck traffic comprises approximately 10 percent of the traffic volume and is not subject to load restrictions in the spring.

The test road is composed of twenty-nine 400-foot length pavement sections, 24 feet wide, with 15 sections constructed on a clay subgrade and 14 sections constructed on a sand subgrade. The types of asphaltic concrete used in the test sections were comprised of various mixes which included two different types and three different grades of asphalt, two asphalt and filler contents, and limestone and granite aggregates. Pavements of three structural designs were incorporated in the test sections. The above variables were selected as being potentially important in the study of transverse pavement cracking and are shown in Table 2 in their respective combinations.

The Pavement Structure Designs

The three different pavement structure designs which were incorporated in the test road are the following:

1. four-inch asphaltic concrete with a 16-inch crushed gravel base course on a clay subgrade (4-16-CLAY) shown in Figure 7,

Table 2. TEST ROAD DESIGN VARIABLES IDENTIFIED BY SECTION NUMBERS

	150-200 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					150-200 PENETRATION GRADE HIGH VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					300-400 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)				SC-5 ASPH. (W.C. CRUDE)
ROAD STRUCTURE	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	ABOVE OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT, 100% CRUSH IGNEOUS AGGREGATE	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT
4 IN. PAVEMENT ⊗ 16 IN. BASE COURSE CLAY SUBGRADE	54	55	63	53		57		62	56	51	58	59	61	60	52
4 IN. PAVEMENT ⊗ 6 IN. BASE COURSE SAND SUBGRADE	74	76	67	75	77	73		66	72	78	71	70	68	69	79
10 IN. FULL DEPTH ⊗ ASPHALT PAVEMENT CLAY SUBGRADE			64					65							

⊗ All aggregates in bituminous pavement mix processed from glacial drift deposits (20% igneous, 80% limestone; 50% crush) unless otherwise indicated.

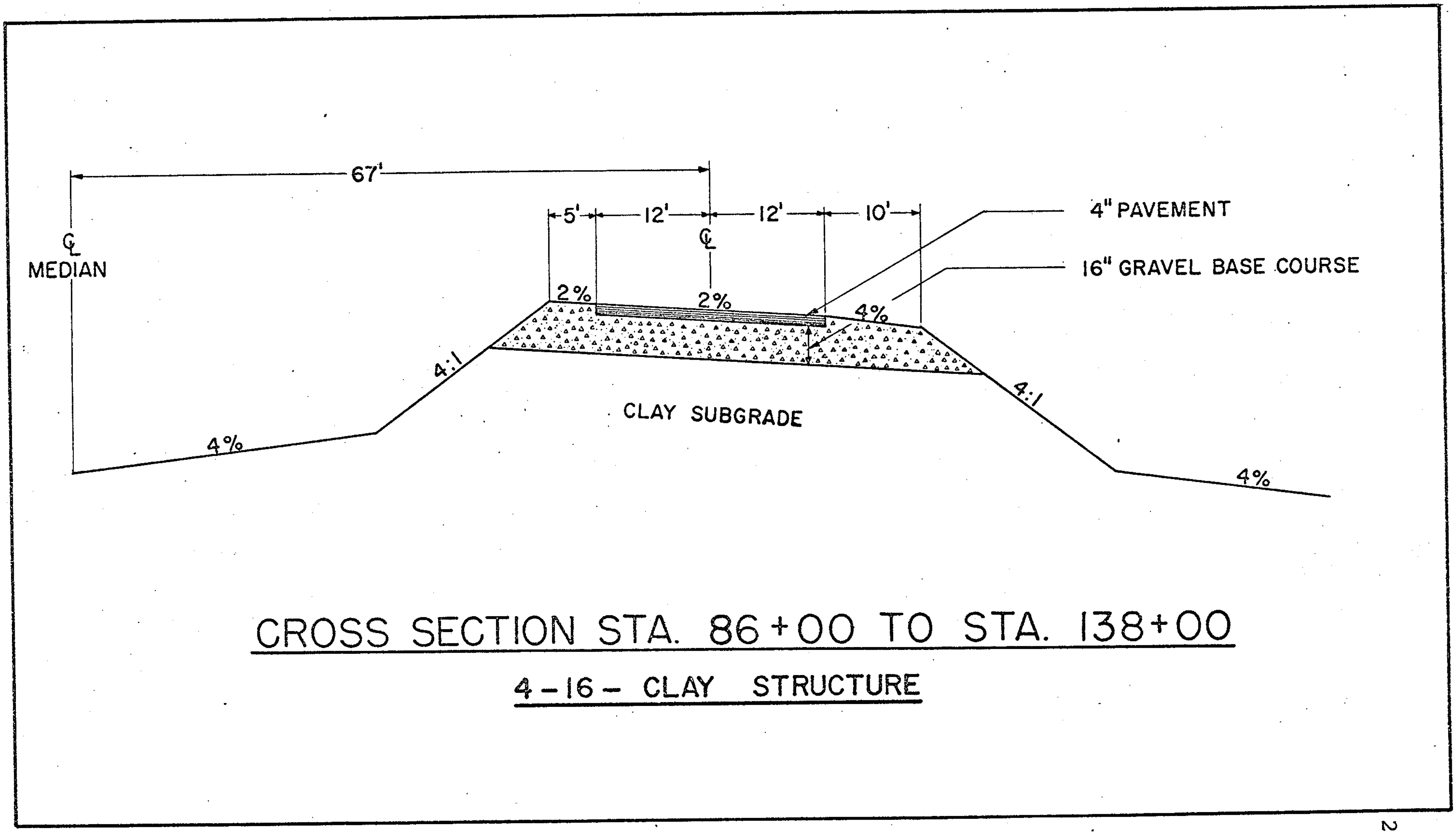


Figure 7. Pavement Cross-Section from Sta. 86+00 to Sta. 138+00

2. ten-inch full depth asphaltic concrete pavement on a clay subgrade (10-0-CLAY) shown in Figure 8,
3. four-inch asphaltic concrete with a six-inch crushed gravel base course on a sand subgrade (4-6-SAND) shown in Figure 9.

A longitudinal section of the above road structures is shown in Figure 10.

The Clay and Sand Subgrades

The 4-16-CLAY and the 10-0-CLAY pavement structures were constructed over a very heavy clay subgrade. The soil type was a fairly uniform A-7 clay with a group index of 20. The liquid limit values were generally in the range of 90 to 105 with the plasticity index ranging from 55 to 65.

Strict control over soil selection was exercised to maintain uniformity of soil type and compaction. As the proposed location of the 10-0-CLAY pavement structure was located in the transition zone between the clay and the sand soils, the existing subgrade was subcut and clay was brought in and placed and compacted to a depth of four feet. A summary of the clay subgrade density and moisture content tests is shown in Appendix I, Table 4.

Several types of tests were conducted to evaluate the bearing capacity of the clay subgrade. The soaked laboratory CBR value was determined to be three percent (AASHO designation T-193-63 with soil compaction procedure modified as per

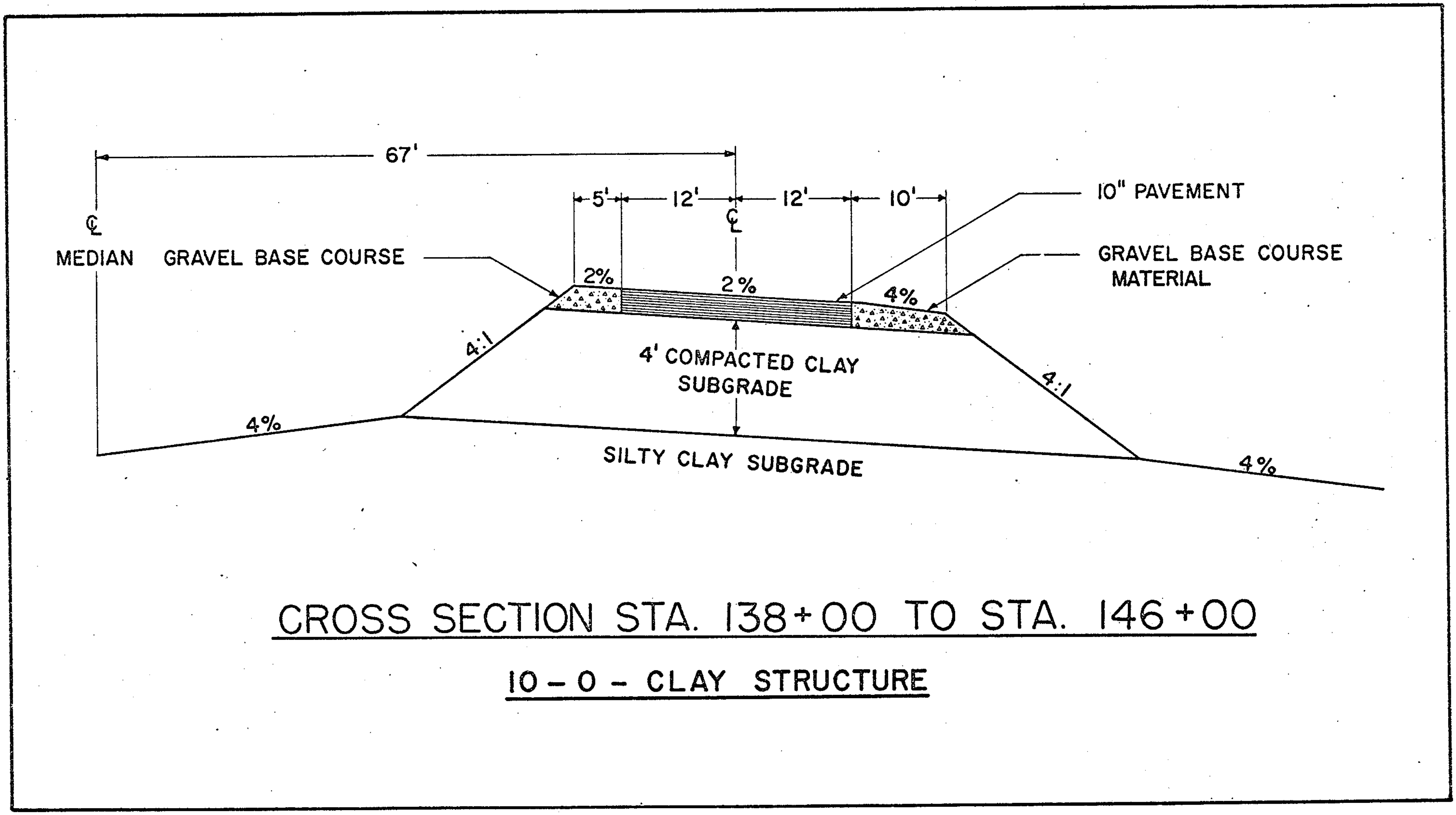
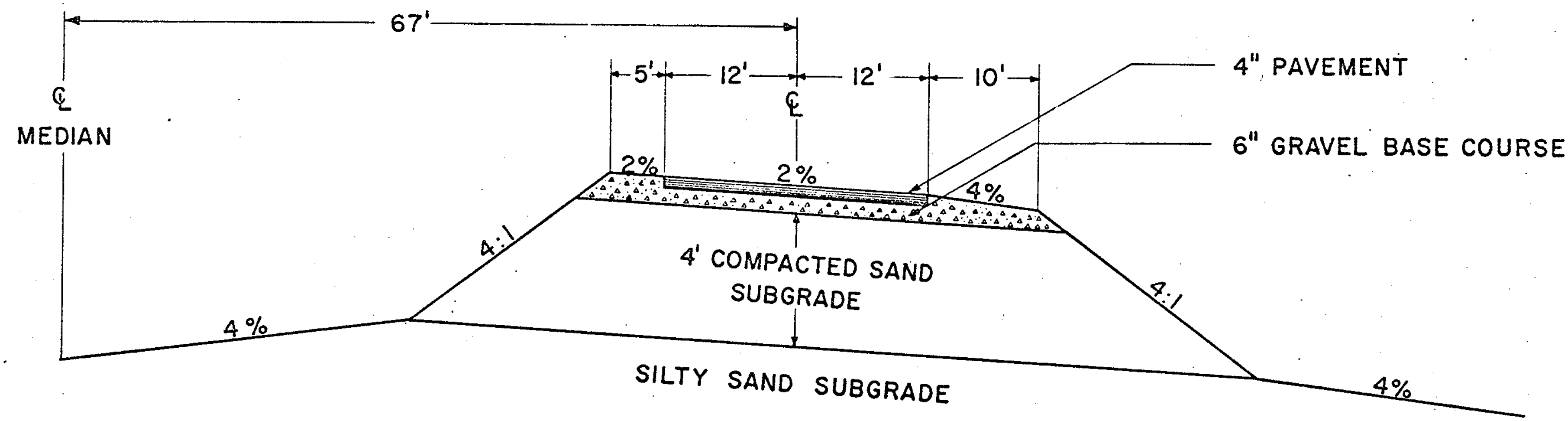


Figure 8. Pavement Cross-Section from Sta. 138+00 to Sta. 146+00



CROSS SECTION STA. 147+00 TO STA. 203+00

4 – 6 – SAND STRUCTURE

Figure 9. Pavement Cross-Section from Sta. 147+00 to Sta. 203+00

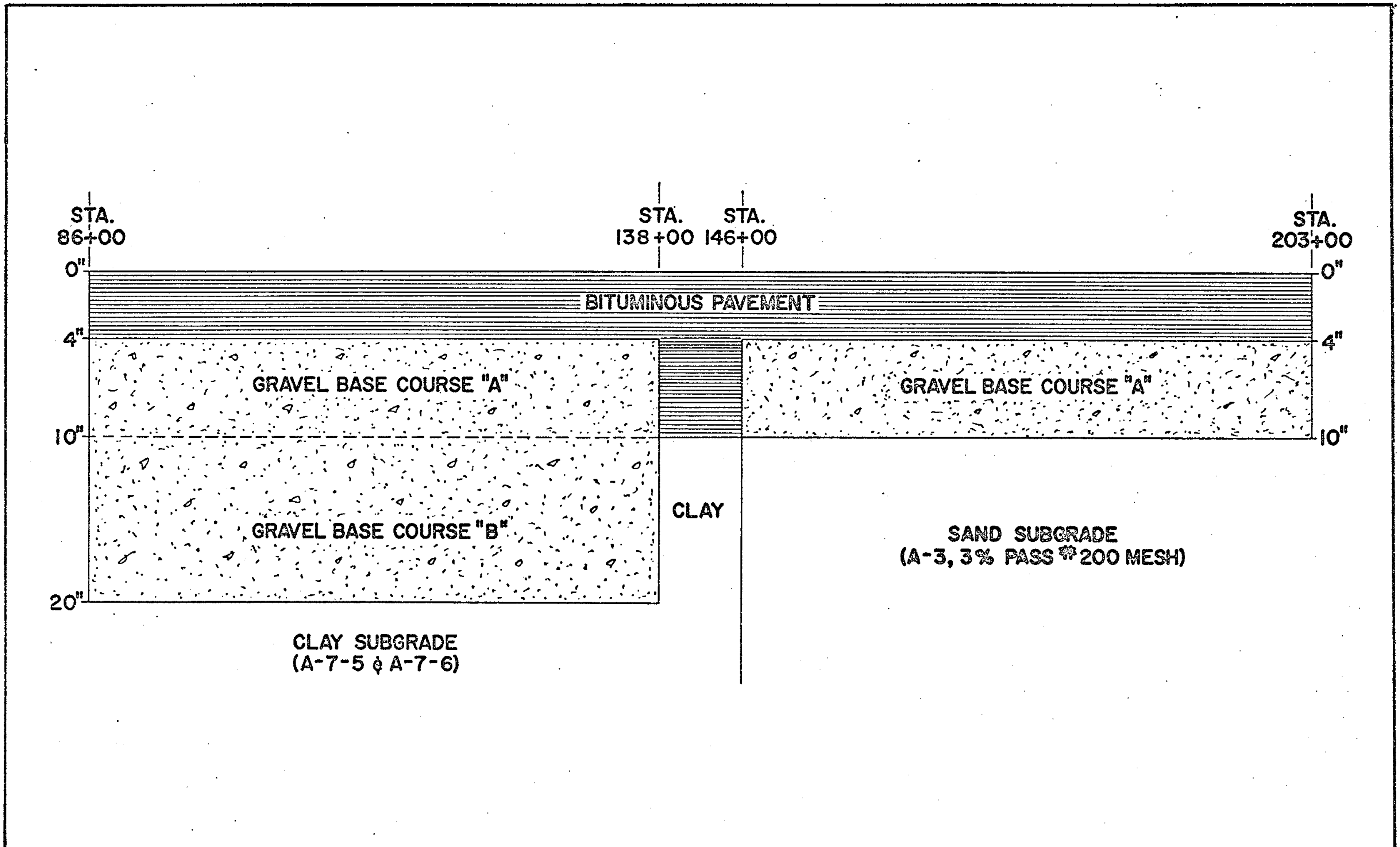


Figure 10. Longitudinal Section of the Ste. Anne Test Road

AASHO designation T-180-61). McLeod plate bearing tests were performed on the compacted clay subgrade and indicated a subgrade support value of 26,600 pounds under a 30-inch diameter plate with 0.5 inch of deflection at 10 repetitions.

The natural sand subgrade was found to be non-uniform, ranging from an A-2 to an A-4 soil type. To insure uniformity, a depth of four feet of uniform A-3 sand with three percent passing the #200 mesh sieve was brought in and compacted. A summary of these densities is found in Appendix I, Table 6.

Evaluations for the bearing capacity of the A-3 sand showed a soaked laboratory CBR value of 50 percent and plate bearing tests of the in-place material indicated a support value equivalent to 108,600 pounds under a 30-inch diameter plate with 0.5 inch of deflection at 10 repetitions.

The Base Courses

The base courses were constructed to the depths shown in Figure 10 from crushed glacial drift deposit material, predominantly a limestone aggregate. The base aggregates were crushed to the gradations shown in Table 7 and compacted to the standards shown in Tables 5 and 6, all appearing in Appendix I.

Soaked laboratory CBR tests showed values of 170 percent for the "B" Base material and 134 percent for the "A" Base material.

The Bituminous Mix Aggregates

Bituminous mixes with two different types of aggregates and two mineral filler contents were among the variables incorporated in the test road in the following combinations:

1. an absorptive aggregate consisting of 80% limestone and 20% igneous aggregate processed from a glacial drift deposit and having a 50% crushed stone content of the material retained on the #4 mesh sieve,
2. the above aggregate with approximately three percent portland cement filler added,
3. a 100% igneous aggregate with 100% crushed particles in all the sieve size ranges and with 5½ percent cement filler comprising the minus #200 mesh fraction.

The aggregates were processed to provide a gradation conforming with Manitoba class "A" 3060 and Swedish AB 16T specification limits. For certain sieve sizes, the specification overlap proved to be too narrow and it was found necessary to exceed the limits in these cases. The summary of the gradations of the extracted mixes appears in Appendix I, Table 8.

The two types of aggregates exhibited different properties. The mean water absorption of the limestone aggregate was in the range of 2.2 to 2.6 percent while the igneous aggregate, consisting predominantly of microcrystalline basaltic and macrocrystalline granitic rock, was less than 0.4 percent. The wear from the Los Angeles abrasion test (ASTM Test Method C136-66) was 26 percent for the limestone aggregate and 14 percent for the igneous aggregate.

Economic considerations did not permit a full factorial design with the igneous aggregate and its use was limited to two 400-foot pavement sections of the test road.

The Asphalts

The asphaltic binder variables encompassed two different types and three different grades of asphalt. The two different types of asphalts are referred to as low viscosity (LV) and high viscosity (HV) types. The three grades of asphalt were 150-200 and 300-400 penetration grades and an SC-5 liquid asphalt. The asphalts used were:

1. low viscosity 150-200 penetration,
2. high viscosity 150-200 penetration,
3. low viscosity 300-400 penetration,
4. high viscosity SC-5.

The variation in the original quality of these products is not indicated in this report and only the test results from representative samples are shown in Appendix I, Table 9.

To evaluate the significance of asphalt content with respect to transverse pavement cracking, the asphalt contents of the mixes were varied in the use of certain of the above binders. The predominating asphalt contents used were:

1. Marshall optimum,

2. one percent below Marshall optimum.

One 400-foot pavement section was constructed with the asphalt content at one-half percent above Marshall optimum.

Pavement Mix Design

Marshall mix design was carried out by Manitoba Highways Department according to ASTM Test Method D 1559-65 using manual compaction at 50 blows per face and by Shell Canada Limited using mechanical compaction at 60 blows per face. The series of test data for the latter is not presented in this report.

The mix design curves for the limestone aggregate mixes are shown in Figures 32, 33, 34 and 35 with the asphalt contents reported on a dry aggregate basis. In all cases where the cement filler was added the design mixes showed an increase in stability, flow, density and voids filled with a reduction in the effective air voids for any given asphalt content. The mix design values based on this data are listed in Appendix I, Table 10.

Pavement Quality Control

To ensure distinct differences among the variables under study, it was essential to minimize the variation in the quality of the individual materials and to exercise strict control over all phases of construction. Where quantity of a product such as asphalt content and cement filler content was a variable under study, it was desirable to minimize the probability variations to an acceptable level to ensure that they constituted different levels of the variables.

Table 11, appearing in Appendix I, illustrates the degree of control attained in the asphalt content and the fines content passing the #200 mesh sieve for the bituminous mixes in each of the sections. A series of statistical tests for the significance of the difference between the means of paired sections was carried out using the t-test and is summarized in Appendix II. They showed that the difference in the mean asphalt content between the lean and the optimum asphalt content mixes, as well as the mean fractions passing the #200 mesh sieve between the filler and the non-filler mixes, were significant at the one percent level. This indicated that the desired objectives with respect to the control of asphalt and mineral filler content had been realized.

The breakdown and intermediate compaction of the test road mixes was carried out with a Bros Air-on-the-Run SP6000 roller and a steel roller was used for final rolling. The pavement densities were measured with a Lane-Wells Road Logger mobile nuclear unit. As the density output was in graphical chart form, it was converted to numerical data with a Calma 480 Digitizing Unit to permit statistical analyses. This yielded a pavement density value for every foot along each lane.

The summary of the densities in Appendix I, Tables 12 and 13 indicates that the degree of pavement compaction achieved ranges from 97 to 100 percent of the standard Marshall design densities appearing in Table 10. For the majority of the sections there does not appear to be any significant difference in the densities between the lanes.

To evaluate the structural uniformity within the test sections, Benkelman beam readings were taken on a weekly basis during 1968. Six readings were taken in the traffic lane of each section. The low standard deviations of the readings during the peak deflection periods, summarized in Appendix I, Table 14, indicate a fair degree of uniformity within sections.

CHAPTER IV

TRANSVERSE CRACK FREQUENCY ANALYSES OF THE STE. ANNE TEST ROAD PAVEMENTS

Introduction

The analyses of the transverse crack frequencies of the Ste. Anne Test Road pavements are based on the transverse cracking observed to date. The data used in the analyses were obtained from pavement crack maps obtained through daily visual observations during the 1967-1968 and 1968-1969 winters and at less frequent intervals during the corresponding summer months. As past experience in Manitoba has shown that low viscosity 150-200 penetration grade asphalt pavements undergo the most severe cracking within the first two winters after construction, it is thought that these sections in the test road will exhibit little additional transverse cracking. However, experience is limited in evaluating the transverse cracking performance of other asphalt types, grades and mix variables, and therefore it cannot be predicted how valid the present evaluation of these variables will be in the years to come.

Methods of Analysis

A study of the vehicle travel characteristics on the

Ste. Anne Test Road has indicated that both automobiles and trucks generally travel in the right-hand lane, permitting a comparison of transverse cracking between a lane subject to traffic loading and an adjoining lane which is relatively free from traffic loading. This travel characteristic has been attributed to the low traffic volume and has been verified by measurements showing a greater amount of rutting and pavement surface wear in the traffic lane than in the passing lane. A distinctively greater amount of pavement surface wear in the wheel-paths of the traffic lane is shown in Figure 11.

To provide a suitable basis for comparing the pavement cracking among the various sections and between the traffic and passing lanes, each lane was studied separately. The transverse cracks were classified into four categories according to a method used by McLeod (15), illustrated in Figure 12.

Separate analyses of variance were carried out for Type 1 cracks and for all crack types combined.

In the study of the transverse cracking frequency of the test road pavements, it was found necessary to resort to the analysis of crack spacing or "slab" length. This approach permitted the evaluation of the nature of the probability distribution of the transverse cracks within a section and provided an estimate of the accuracy and precision of the mean of the spacing between cracks.

Figure 11. Pavement Wear in the Wheelpaths of the Traffic Lane.

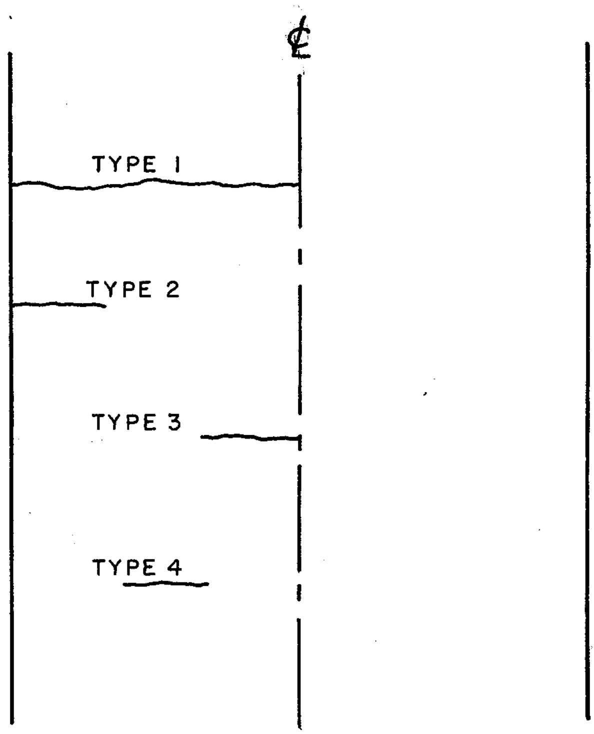


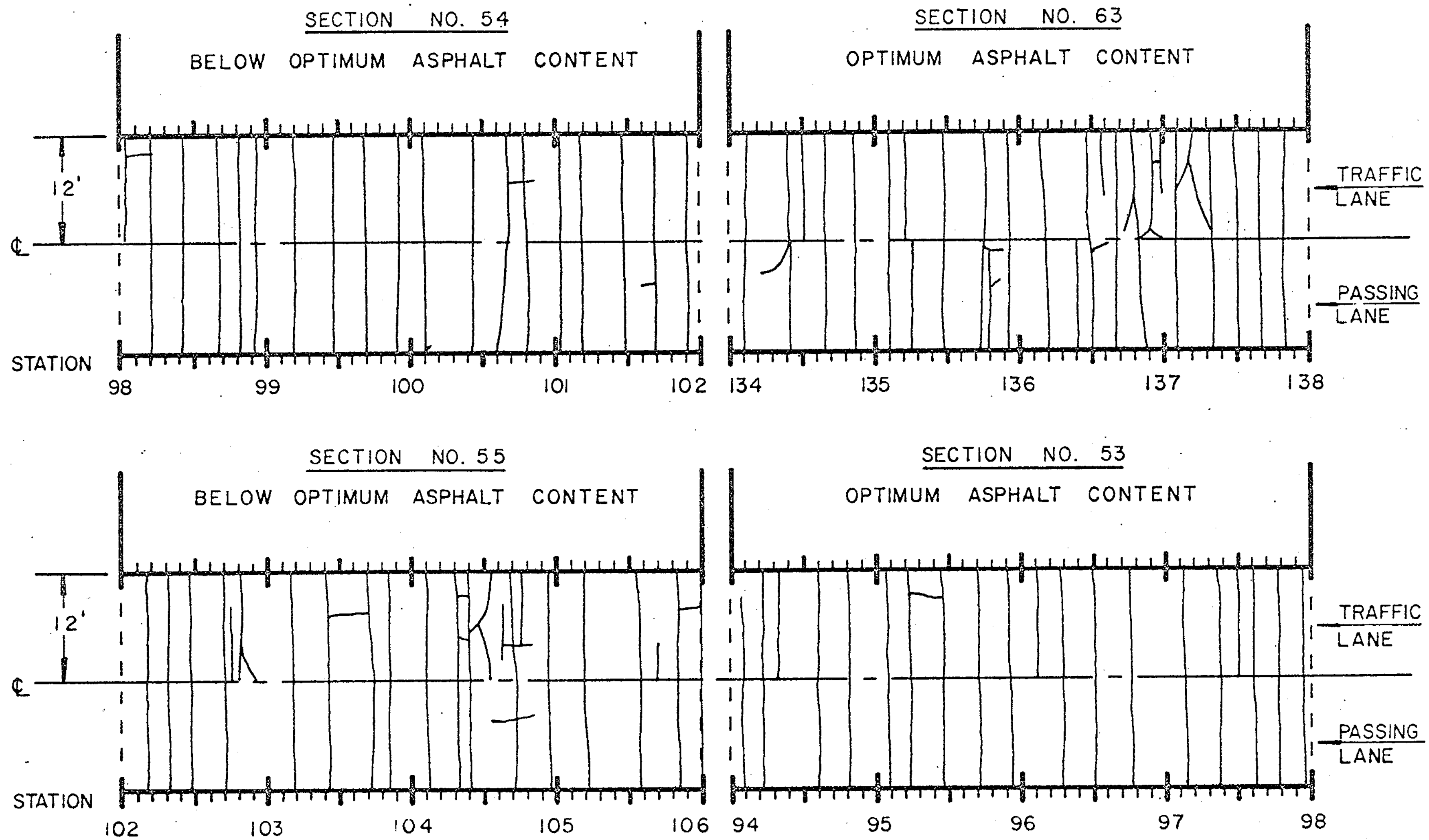
Figure 12. Types of Transverse Pavement Cracks.

Statistical comparisons were carried out (Appendix II) to determine whether the observed differences in mean transverse crack spacing between traffic lanes and passing lanes, between sections and among sections were significant or whether the dispersion of data could be attributed to chance distribution. Significance levels of the relationships were established using the F-distribution to test the variances of groups of data and Student's t-distribution to test the differences between the means.

<u>Transverse Cracking of LV 150-200 PEN. Asphalt Pavements</u>

Figure 13 illustrates the pattern of transverse cracking in the LV 150-200 PEN asphalt 4-16-CLAY pavement sections. Almost all of the transverse cracking may be classified as falling into the Type 1 category. Statistical analyses in Appendix II indicated that:

1. there is no significant difference in the frequency of transverse cracking between the traffic and passing lanes (Table 25),
2. the asphalt content is not significant in affecting transverse cracking frequency (Table 15),
3. the cement filler content, in the range studied, is not significant in its effect on transverse cracking frequency (Table 16),
4. the mean of the spacing between cracks is 20 ± 5 feet or an average of 264 transverse cracks per



TRANSVERSE PAVEMENT CRACKING TO MARCH 31,1969

ASPHALT: 150-200 PEN. LOW VISCOSITY AGGREGATE : LIMESTONE
STRUCTURE : 4 - 16 - CLAY

Figure 13. Transverse Cracking of LV 150-200 PEN., 4-16-CLAY Pavements

mile (95% confidence level, Table 27).

The latter statistic compares well with the findings of a 1964 survey of 312 miles of pavements in Manitoba of various structural designs with clay subgrades. These 150-200 penetration grade asphalt pavements, predominantly of the low viscosity asphalt type and ranging in age from one to ten years, exhibited a mean spacing between cracks of 25 feet or an average of 211 transverse cracks per mile. The frequency diagram compiled from the survey data is shown in Figure 2.

Figures 14 and 15 illustrate the transverse cracking of the LV 150-200 PEN. asphalt 4-6-SAND pavements. Comparisons of transverse cracking between the traffic and the passing lanes of the sections indicated no significant difference when Type 1 cracks were considered and one section out of five indicated a significant difference at the five percent level but not at the one percent level when all types of transverse cracks were considered. Thus, it is concluded that traffic loading was not significant in contributing to transverse pavement cracking.

The data in Table 27 indicates that the mean of the spacing between cracks at the 95 percent confidence level for all of the LV 150-200 PEN. asphalt 4-6-SAND pavements was 8 ± 5 feet (660 cracks/mile) for Type 1 cracks and 6 ± 4 feet (880 cracks/mile) when all types of transverse cracks were considered. However, the difference among the

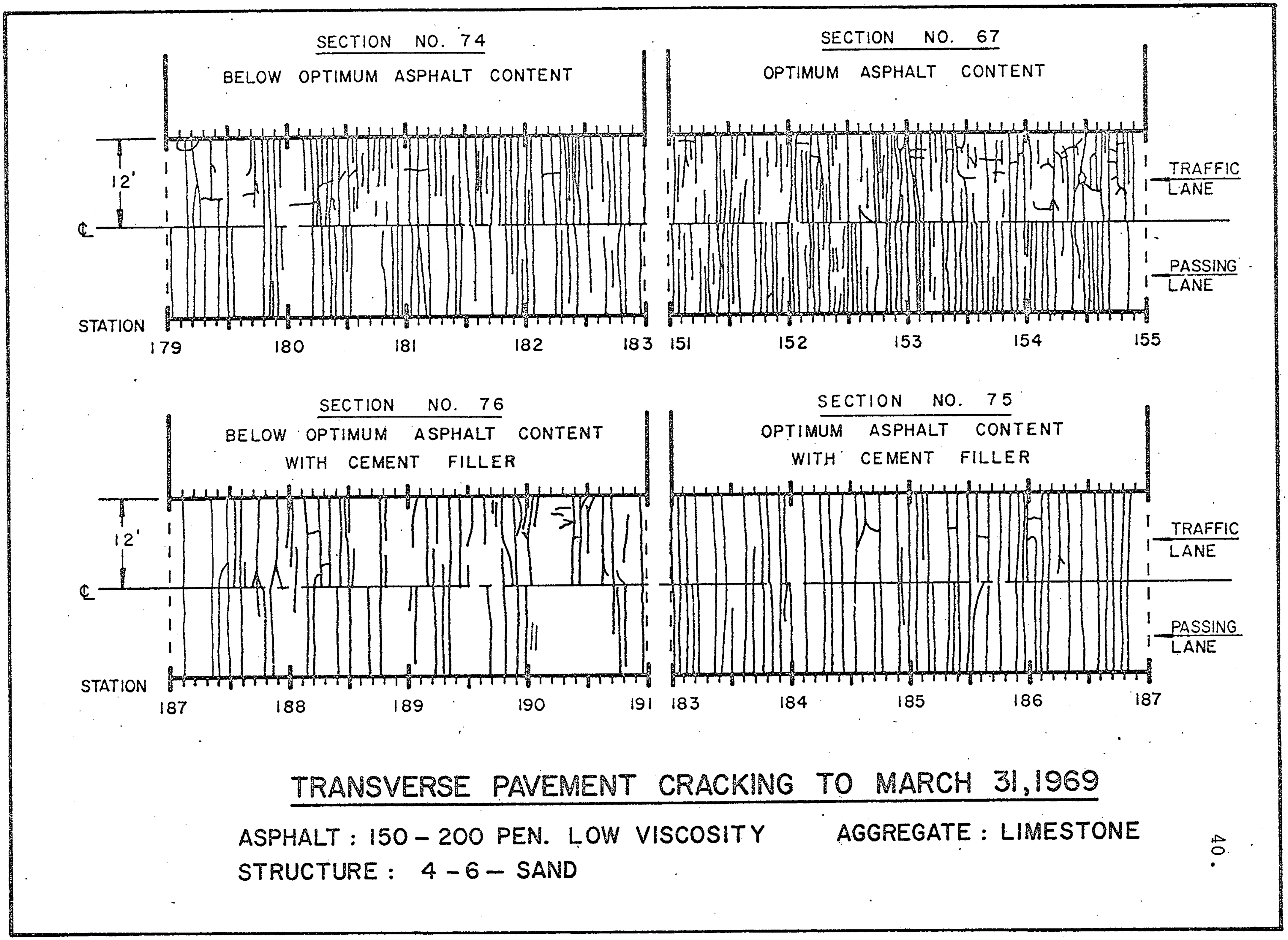
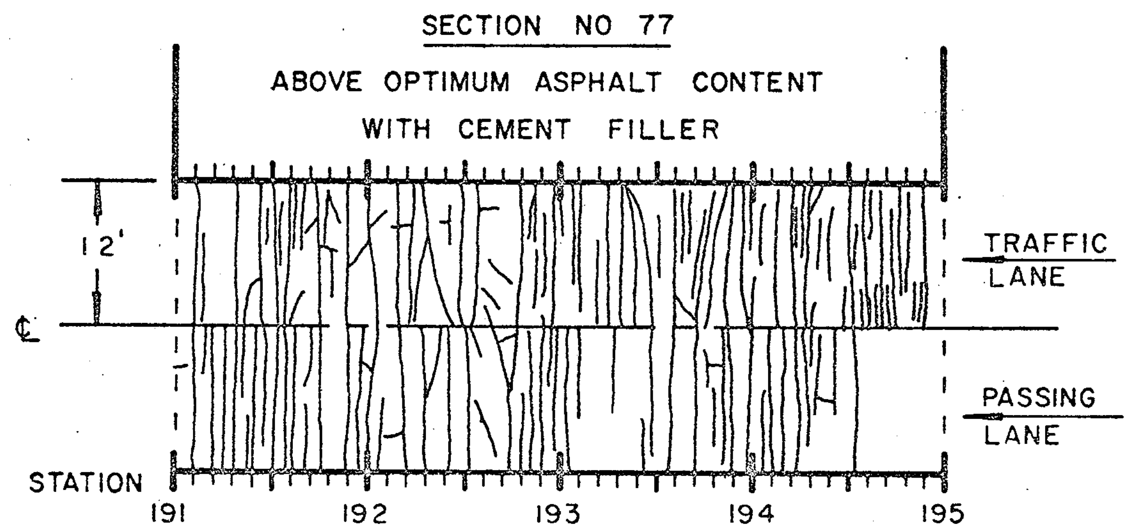


Figure 14. Transverse Cracking of LV 150-200 PEN., 4-6-SAND Pavements



TRANSVERSE PAVEMENT CRACKING TO MARCH 31, 1969

ASPHALT: 150 – 200 LOW VISCOSITY AGGREGATE : LIMESTONE

STRUCTURE : 4 – 6 – SAND

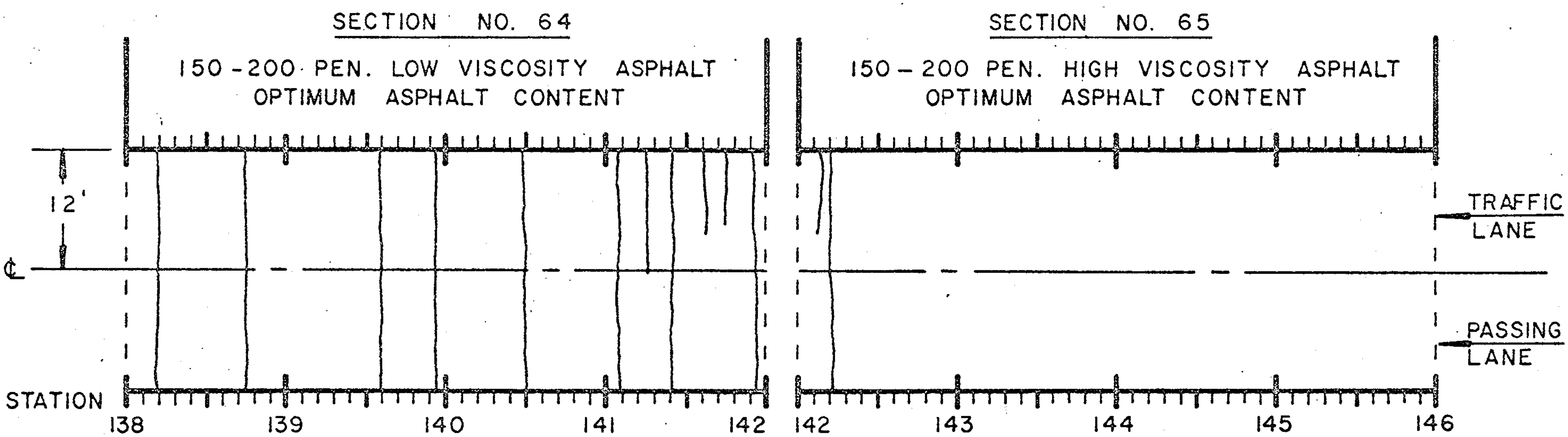
Figure 15. Transverse Cracking of LV 150-200 PEN., 4-6-SAND Pavement in Section No. 77

means did not reflect any increase or decrease in transverse cracking frequency from varying the asphalt content and the cement filler content in the asphaltic concrete mixes. Thus, neither asphalt content nor cement filler content was significant in affecting the frequency of transverse pavement cracking within the limits of the ranges studied.

The difference between the means of the transverse crack spacing of the 4-16-CLAY and 4-6-SAND pavements was found to be significant at the one percent level. A comparison of crack frequencies indicated that the pavements with a sand subgrade exhibited 2½ times more Type 1 cracking and three times more cracking of all types than pavements with similar asphaltic concrete mixes constructed in a clay subgrade area. Thus, there is an interaction between the asphaltic concrete and the subgrade at low temperatures which affects the frequency of transverse cracking of the pavement.

The transverse cracking of the LV 150-200 PEN. asphalt 10-0-CLAY pavements is shown in Figure 16. The transverse cracking between Station 141+50 and 142+00 was attributed to construction joints and was not included in the analyses. The mean of the crack spacing is 50 ± 10 feet (106 cracks/mile) at the 95 percent confidence level.

This indicates that the ten-inch full depth asphalt pavement built on a clay subgrade (10-0-CLAY) exhibited 40 percent of the transverse cracking observed in the 4-16-CLAY



TRANSVERSE PAVEMENT CRACKING TO MARCH 31, 1969

STRUCTURE : 10 – 0 – CLAY(FULL DEPTH PAVEMENT) AGGREGATE : LIMESTONE

Figure 16. Transverse Cracking of 10-0-CLAY Pavements

pavements and only 16 percent of the Type 1 cracking found in the 4-6-SAND pavements. On this basis it is implied that the frequency of transverse pavement cracking varies inversely to the thickness of the asphaltic concrete.

The significance of the interrelationships remains to be evaluated in terms of its influence on the present serviceability ratings of the pavements. A series of crack width measurements in February 1969 revealed that the crack opening was inversely proportional to the crack frequency. In the 4-6-SAND structure sections, 19 percent of the cracks were less than 1/32 inch in width and remaining cracks had a mean width of 5/32 inch. In the 4-16-CLAY structure sections, 11 percent of the cracks were less than 1/32 inch in width while the remaining cracks had a mean width of 7/32 inch. All of the cracks could be measured in the 10-0-CLAY section and were found to have a mean crack width of 9/32 inch. It is anticipated that the crack opening may have a bearing on the quantity of surface moisture entering the subgrade which may eventually affect the pavement performance.

Transverse Cracking of LV 300-400 PEN. Asphalt Pavements

The transverse cracking of the LV 300-400 PEN. asphalt 4-16-CLAY pavements is shown in Figure 17. The spacing between cracks is generally greater than was found with the LV 150-200 PEN. asphalt 4-16-CLAY pavements. However, in Sections No. 58 and 60 the crack distribution is not at

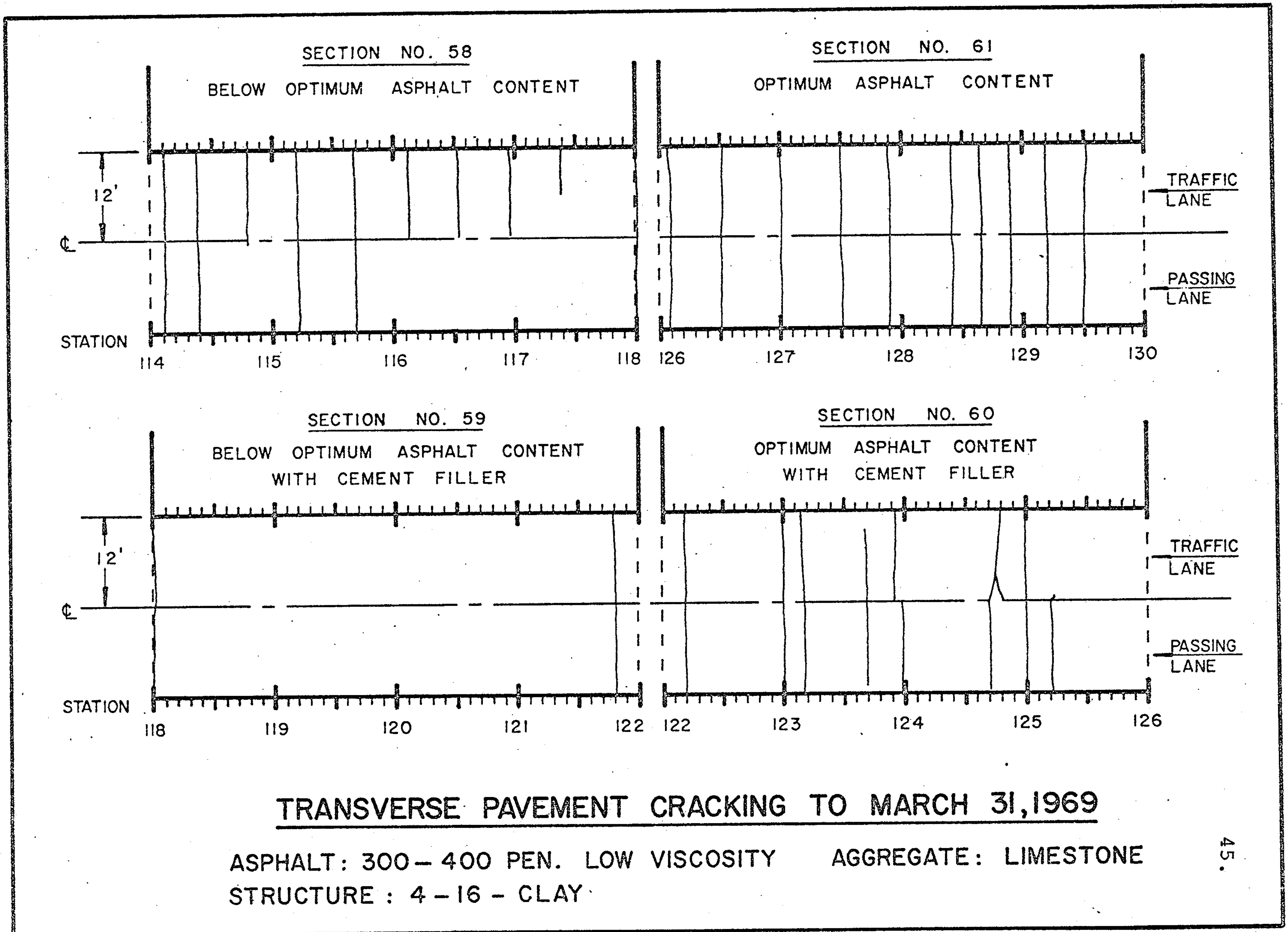


Figure 17. Transverse Cracking of LV 300-400 PEN., 4-16-CLAY Pavements

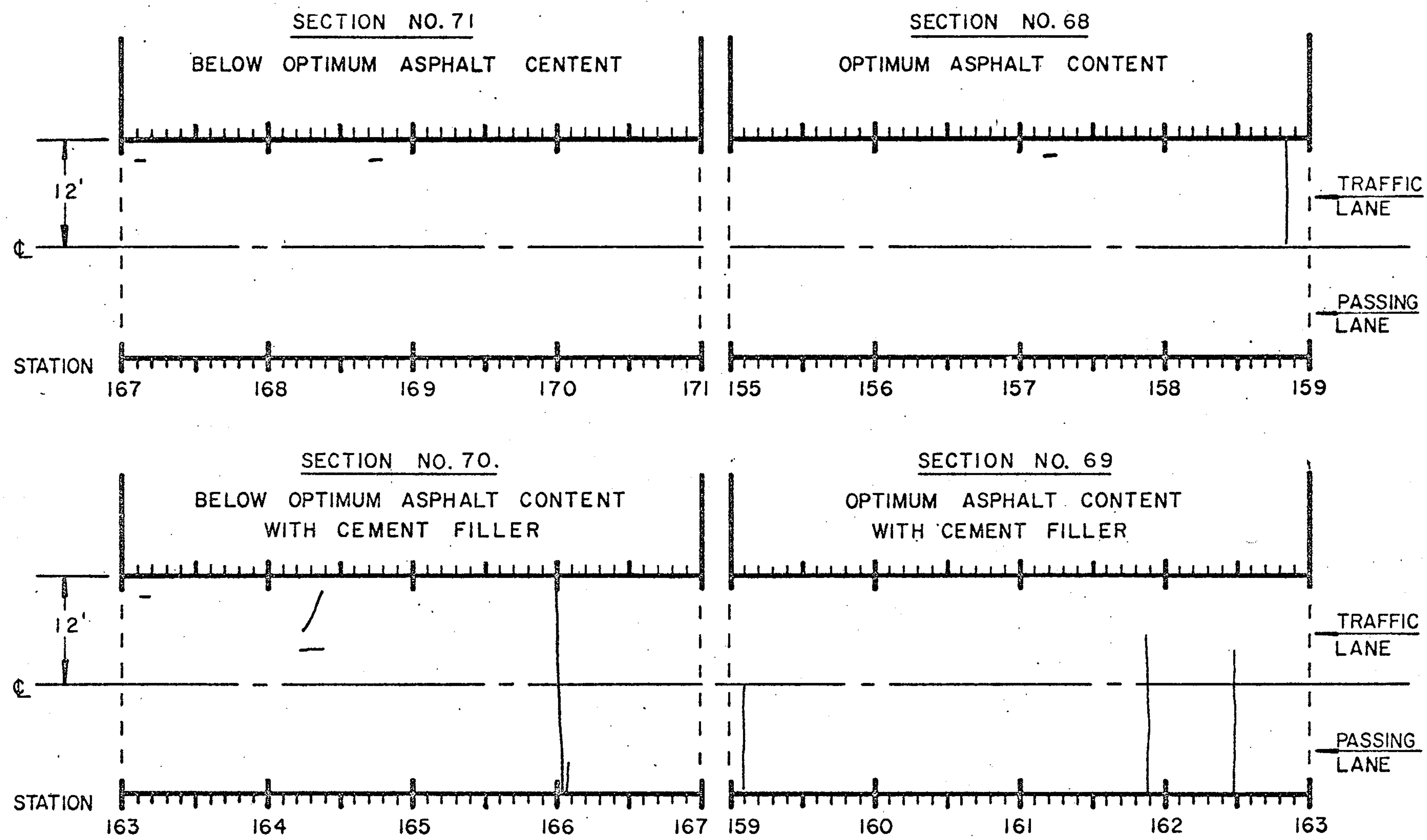
regularly spaced intervals, in Section No. 58 there is a significant difference in cracking frequency between the traffic and the passing lanes, and there appears to be a disparity in the frequency of transverse cracking among the sections which cannot be related to any of the pavement variables under test. For some sections the standard deviation of the crack distribution is abnormally high indicating that the predetermined section length of 400 feet was insufficient to permit the observation of the required number of cracks necessary to draw a reliable conclusion regarding the transverse cracking performance of this asphalt in the 4-16-CLAY pavements.

As asphalt and cement filler contents did not prove to be significant in affecting transverse cracking frequency in the LV 150-200 PEN. asphalt pavements, a similar relationship was assumed to exist for the LV 300-400 penetration 4-16-CLAY pavements. The four adjoining sections were pooled together and the mean spacing between cracks was found to be 65 feet (81 cracks/mile). However, the data appeared too erratic to estimate a meaningful accuracy of the mean. If this is accepted as being representative for the LV 300-400 PEN. asphalt 4-16-CLAY pavements, it is implied that this, the 'softer' grade, exhibits only 30 percent of the transverse cracking observed in the 'harder' LV 150-200 PEN. asphalt 4-16-CLAY pavements.

The LV 300-400 PEN. asphalt 4-6-SAND pavements, shown in Figure 18, exhibited a maximum of only one or two cracks per 400-foot section, demonstrating superior performance in relation to the duplicate sections in the clay subgrade area and with respect to the LV 150-200 PEN. asphalt 4-6-SAND pavements which had a mean spacing of eight feet between Type 1 cracks. This is the opposite of the transverse crack frequency relationship that was found to exist between the two types of subgrades for the LV 150-200 PEN. asphalt pavements; the pavements with sand subgrades exhibited a greater frequency of transverse cracking than those with clay subgrades. This implies that there is an inter-action between the asphaltic concrete surface and the subgrade, that the frequency of transverse cracking is dependent upon the low temperature properties of both the asphaltic concrete and the subgrade, and that the transverse cracking frequency of an asphaltic concrete surface may be altered greatly through the selection of the grade of the asphalt to be incorporated in the mix.

Transverse Cracking of HV 150-200 PEN. Asphalt Pavements

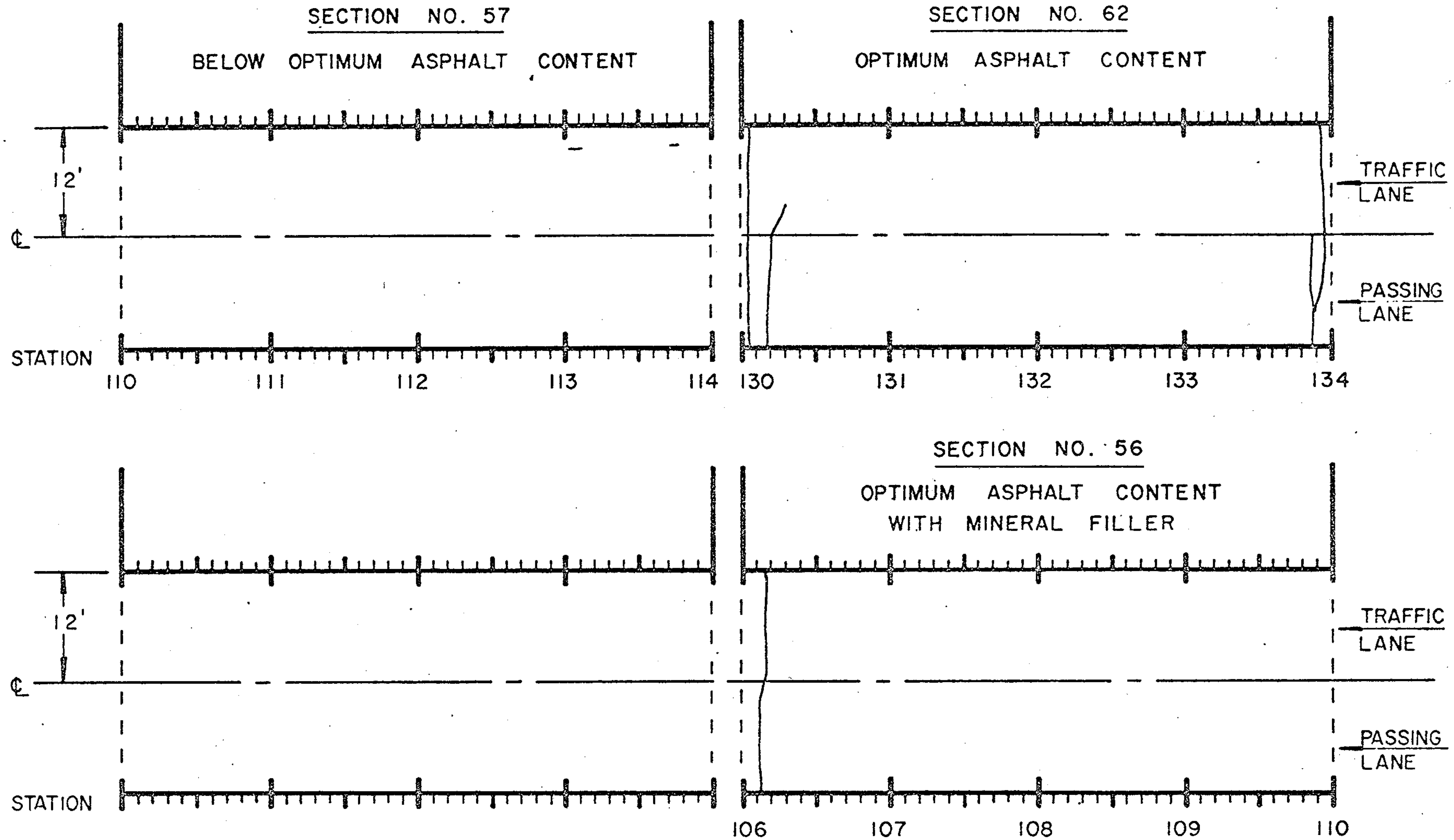
The HV 150-200 PEN. asphalt sections have not exhibited transverse cracking to date in the 4-16-CLAY, 10-0-CLAY, or the 4-6-SAND pavements. The transverse cracks appearing in Figures 19 and 20 are located at construction joints of abutting sections of different asphalts. In comparison to the low viscosity 150-200 penetration grade



TRANSVERSE PAVEMENT CRACKING TO MARCH 31,1969

ASPHALT: 300 – 400 PEN. LOW VISCOSITY AGGREGATE: LIMESTONE
STRUCTURE : 4 - 6 - SAND

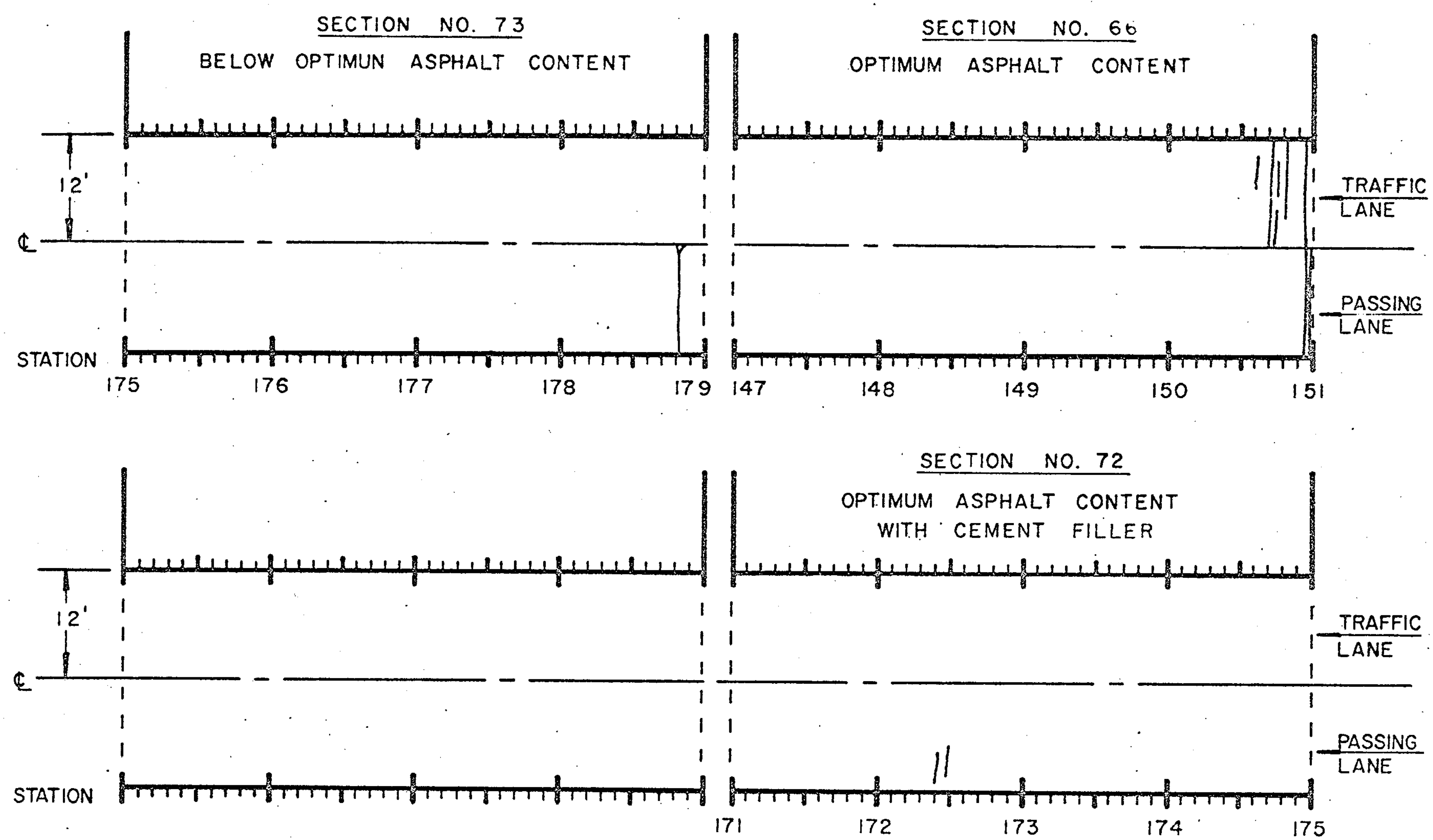
Figure 18. Transverse Cracking of LV 300-400 PEN., 4-6-SAND Pavements



TRANSVERSE PAVEMENT CRACKING TO MARCH 31, 1969

ASPHALT: 150-200 PEN. HIGH VISCOSITY AGGREGATE: LIMESTONE
STRUCTURE: 4-16-CLAY

Figure 19. Transverse Cracking of HV 150-200 PEN., 4-16-CLAY Pavements



TRANSVERSE PAVEMENT CRACKING TO MARCH 31, 1969

ASPHALT: 150 – 200 PEN. HIGH VISCOSITY   AGGREGATE : LIMESTONE
STRUCTURE : 4 – 6 – SAND

Figure 20. Transverse Cracking of HV 150-200 PEN., 4-6-SAND Pavements

asphalt, the high viscosity type of asphalt exhibited superior performance. This indicates that the problem of transverse cracking of flexible pavements can be alleviated through the selection of the proper type and grade of asphalt to be incorporated in the mix.

A direct comparison between the effects of limestone and igneous mix aggregates on transverse cracking cannot be made because a full factorial experiment was not implemented in this respect. The igneous aggregate was used solely in the HV 150-200 PEN. asphalt mixes which did not exhibit any transverse cracking, as shown in Figure 21. As neither the HV 150-200 PEN. asphalt igneous nor limestone aggregate mixes exhibited any transverse cracking while the LV 150-200 PEN. asphalts limestone mixes exhibited a high cracking frequency, it may be concluded that the asphalt is the dominant variable having a significant bearing on the transverse cracking of pavements.

Transverse cracking of HV SC-5 Asphalt Pavements

The HV SC-5 asphalt sections constructed in both the clay and sand portions of the test road have exhibited no transverse cracking to date, as shown in Figure 21. It stands to reason that if two products, a 'softer' grade asphalt (300-400 PEN.) and a higher viscosity asphalt (HV 150-200 PEN.) both exhibit superior performance in comparison to a 'harder' grade low viscosity type of product

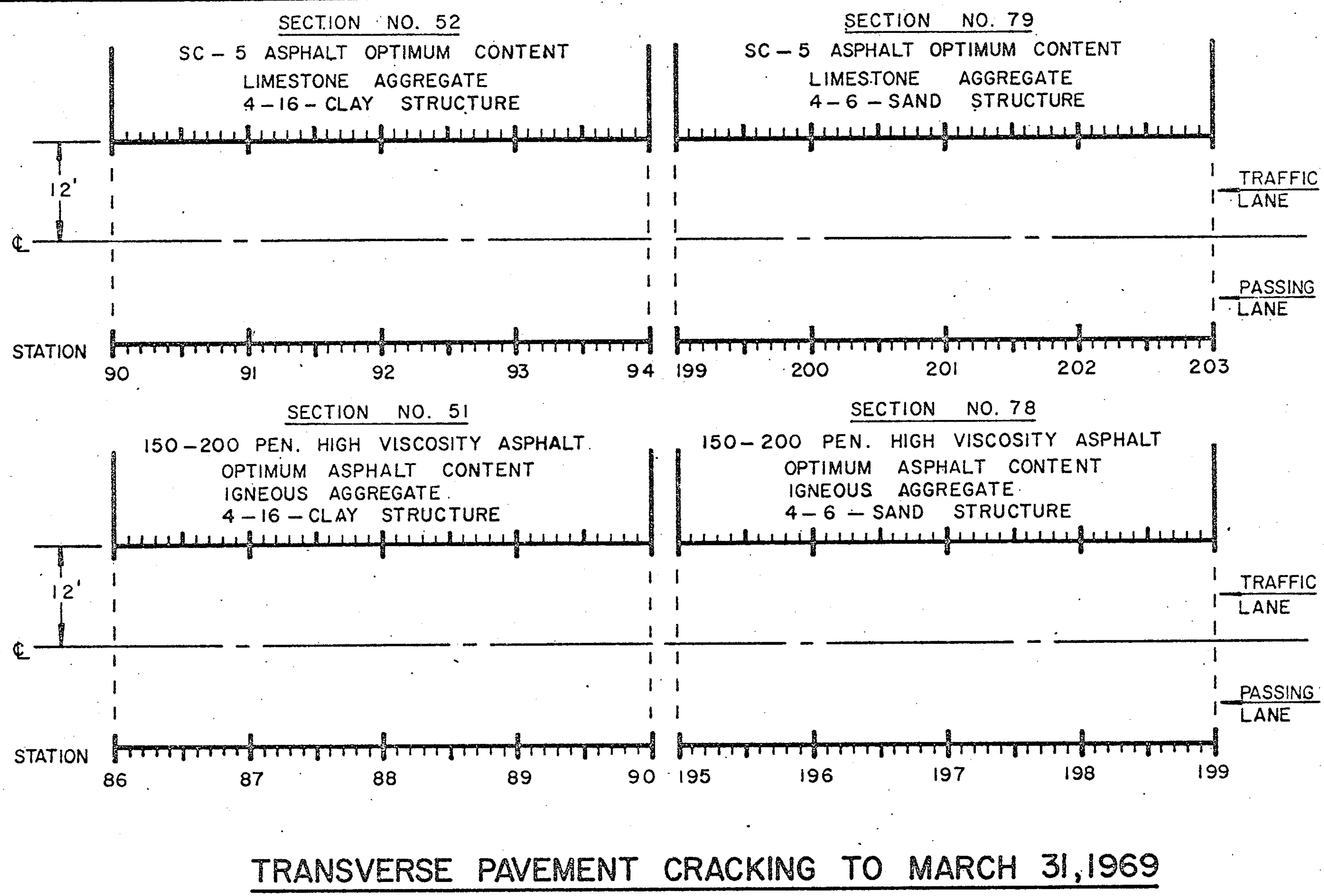


Figure 21. Transverse Cracking of SC-5 Pavements and HV 150-200 PEN., Igneous Aggregate Mix Pavements

(LV 150-200 PEN.), then an asphalt such as the HV SC-5, incorporating both of these desirable properties, will exhibit, at the very least, resistance to transverse cracking comparable to that of the best performer.

Summary of Transverse Crack Frequency Data

A summary of the mean Type 1 crack frequencies observed in the Ste. Anne Test Road pavements to date appears in Table 3.

Table 3. Frequency of Type 1 Cracking of Test Sections

(Number of Cracks Per Mile)

ROAD STRUCTURE	150-200 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					150-200 PENETRATION GRADE HIGH VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)					300-400 PENETRATION GRADE LOW VISCOSITY ASPHALT (WESTERN CANADIAN CRUDE)				SC-5 ASPH. (W.C. CRUDE)
	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	ABOVE OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT, 100% CRUSH IGNEOUS AGGREGATE	BELOW OPTIMUM ASPHALT CONTENT	BELOW OPTIMUM ASPHALT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT, CEMENT FILLER	OPTIMUM ASPHALT CONTENT
4 IN. PAVEMENT 16 IN. BASE COURSE CLAY SUBGRADE ⊕	251	271	285	235		0		0	0	0	64	0	143	119	0
4 IN. PAVEMENT 6 IN. BASE COURSE SAND SUBGRADE ⊕	704	480	1056	621	621	0		0	0	0	0	0	0	0	0
10 IN. FULL DEPTH ASPHALT PAVEMENT CLAY SUBGRADE ⊕			107					0							

⊕ All aggregates in bituminous pavement mix processed from glacial drift deposits (20% igneous, 80% limestone; 50% crush) unless otherwise indicated.

CHAPTER V

OBSERVATIONS OF THE MECHANISM OF TRANSVERSE CRACKING

Temperature Conditions at Initiation of Transverse Cracking

To gain insight into the nature of the transverse cracking mechanism, it was essential to define the thermal regime of the Ste. Anne Test Road pavements at the time of initiation of transverse cracking. An automatic transverse crack detection system was implemented in seven of the 29 pavement sections (13). Long strips of electrical conductor foil tape were embedded into the pavement and the circuits were connected to continuously monitoring recorders. Visual crack surveys were also conducted during the 1967-1968 and 1968-1969 winters on a daily basis and at less frequent intervals during the corresponding summer months.

The temperatures within the pavement structures were measured by thermocouples and printed out at half-hourly intervals by automatic recorders. Banks of thermocouples, at various depths, were situated in each of the three pavement structures. The sand subgrade portion of the test road was located in a wooded area, sheltered from the direction of the prevailing wind during the winter months, while the clay subgrade portion was situated on prairie, completely void of trees. A study of the temperature data revealed that the temperatures at corresponding locations near the pavement surface were a few degrees lower in the pavements with clay subgrades than in the pavements with sand subgrades. This difference was attributed to a cooling of the pavement by the wind in the unsheltered clay subgrade area of the test road.

All of the transverse cracks observed to date initiated during the 1967-1968 winter, the first winter after the construction of the test road. No further transverse cracking occurred during the 1968-1969 winter. Figures 22, 23, 24 and 25 illustrate the relationship of the incidence of transverse cracking to the minimum daily temperatures within the pavement structure and indicate that most of the transverse cracking resulted from the effects of a prolonged low temperature during which the asphaltic concrete cooled gradually throughout its thickness and reached a low temperature level.

Data from the temperature recorders indicated that a difference in the temperatures of the asphaltic concrete existed at various depths and that the temperature at any depth was subject to daily variations. Data from temperature recorders and transverse crack detection circuits indicated that during the December 17, 1967 to January 11, 1968 low temperature period the pavement temperature intensities at initiation of transverse cracking were

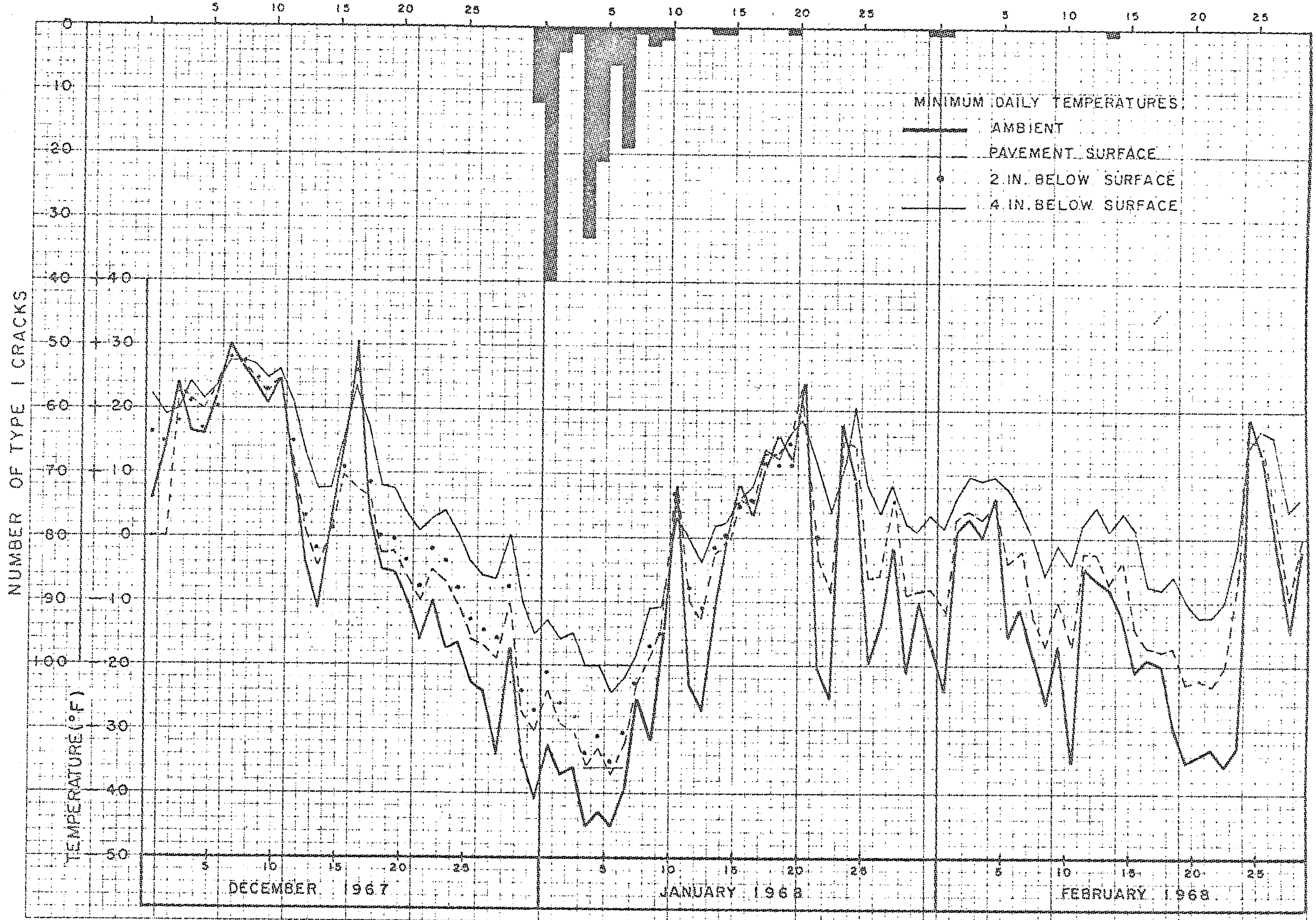


FIGURE 22 — INITIATION OF TRANSVERSE CRACKING IN SECTIONS 53,54,55 & 63 (LV 150-200 PEN., 4-16-CLAY)

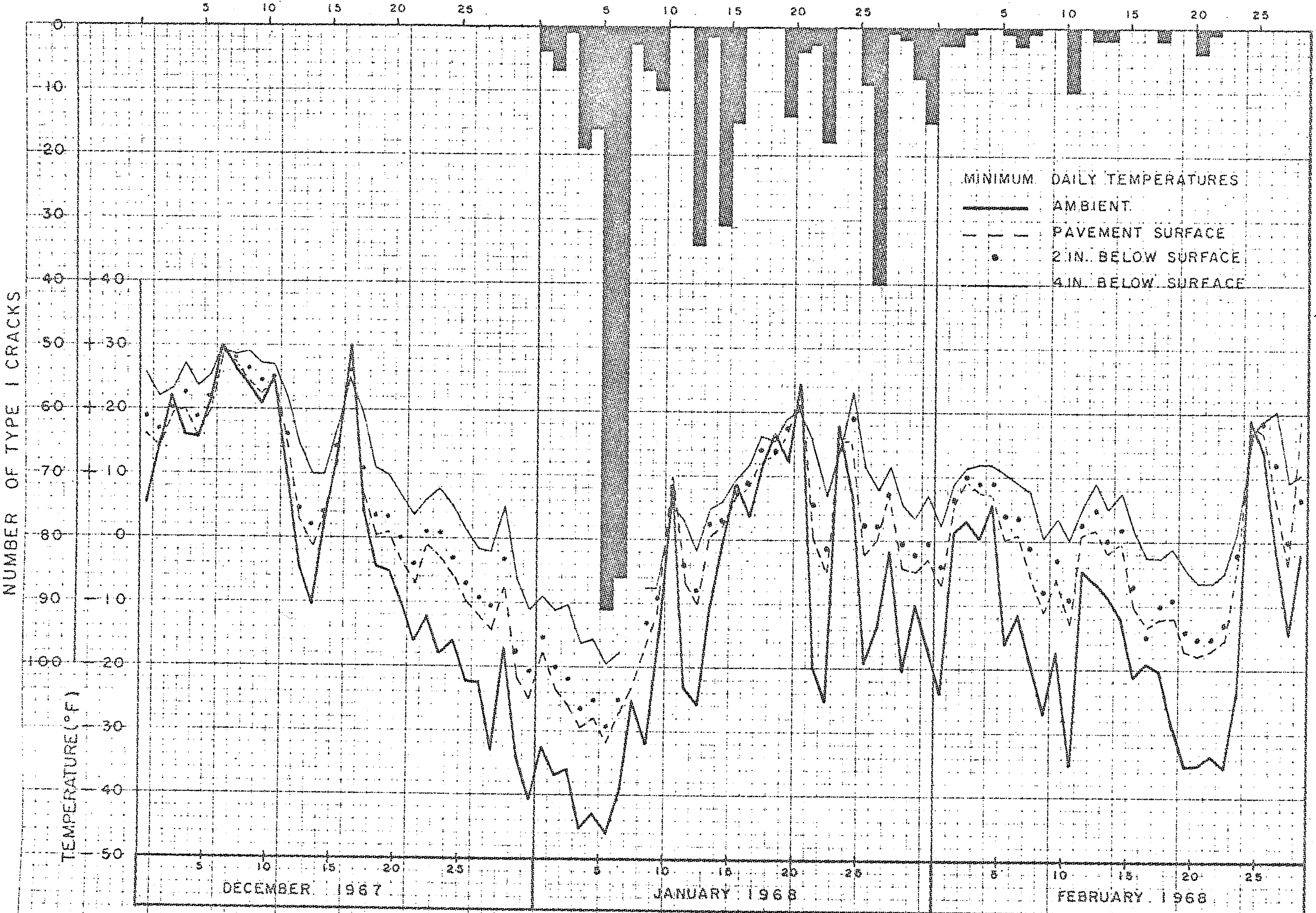


FIGURE 23 — INITIATION OF TRANSVERSE CRACKING IN SECTIONS 67,74,75,76 & 77 (LV 150-200 PEN., 4-6-SAND)

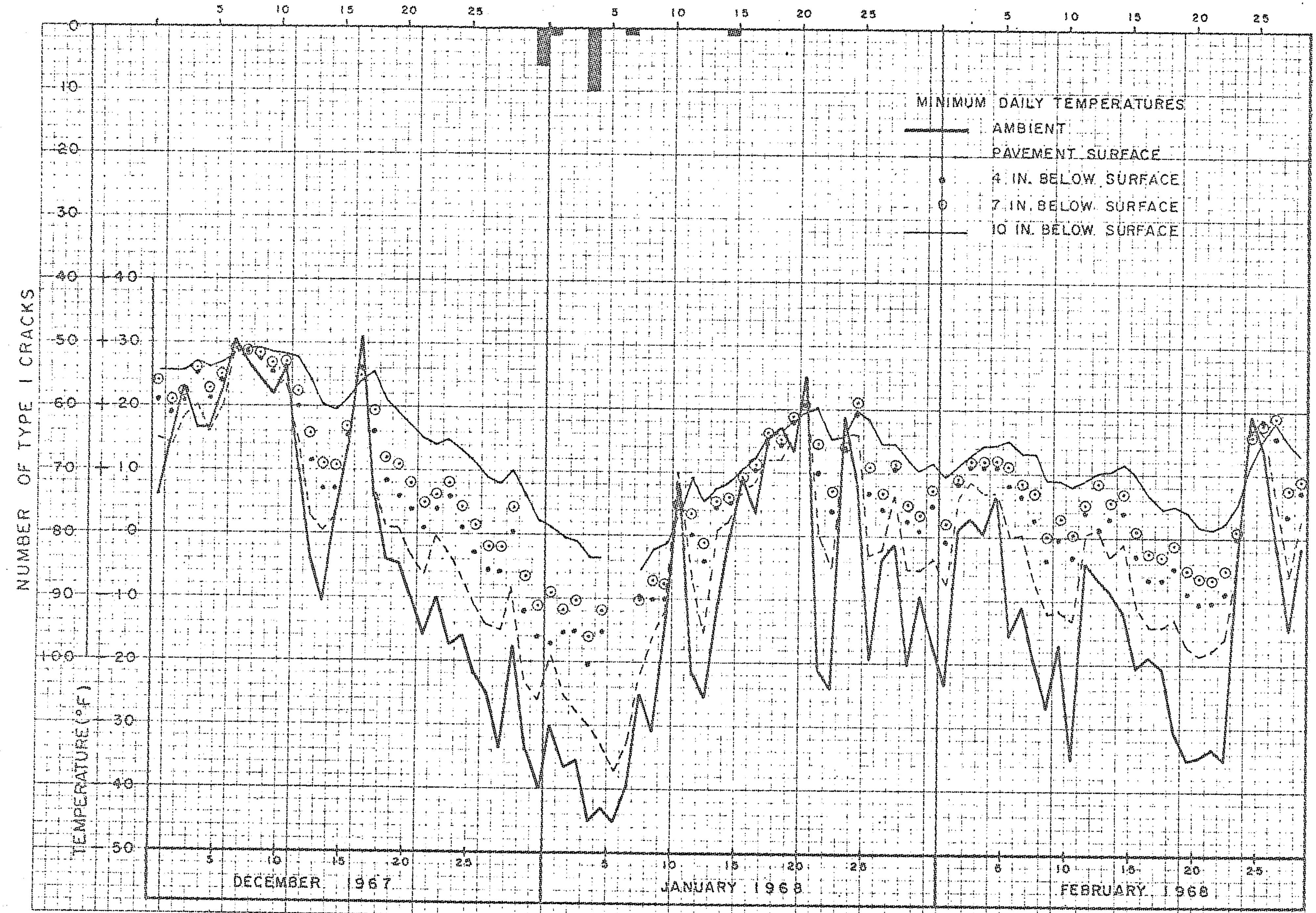


FIGURE 24 — INITIATION OF TRANSVERSE CRACKING IN SECTION 64 (LV 150-200 PEN., 10-0-CLAY)

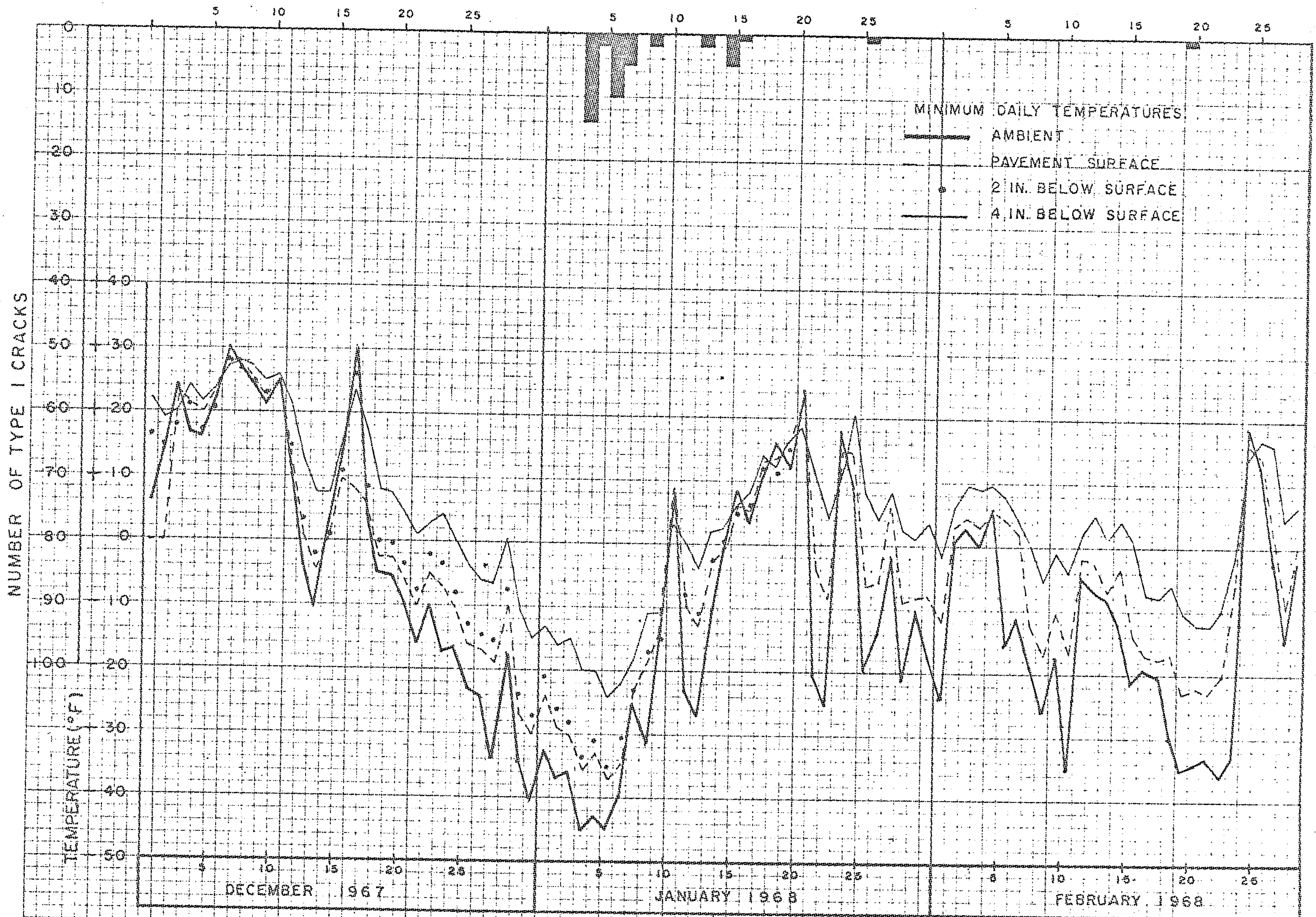


FIGURE 25 — INITIATION OF TRANSVERSE CRACKING IN SECTIONS 58, 60 & 61 (LV 300-400 PEN., 4-16-CLAY)

generally within two to six degrees of the minimum daily temperature at the corresponding depth. Thus, the minimum daily temperatures provided a suitable basis for estimating the pavement temperature intensities at the initiation of transverse cracking for all the pavements which were not instrumented with transverse crack detection circuits but for which daily visual crack survey data was available.

The first transverse cracks in the LV 150-200 PEN., 4-16-CLAY pavements were observed on December 31, 1967, after the temperatures had reached intensities on the previous night of -30° F at the pavement surface, -27° F at a depth of two inches below the surface and -15° F at a depth of four inches below the surface, as shown in Figure 22. Thereafter, the majority of the transverse cracking occurred between January 4 and 7, 1968 when the minimum daily temperatures ranged from -33° F to -37° F at the pavement surface, -30° F to -35° F at a depth of two inches below the surface and -20° F to -24° F at a depth of four inches below the pavement surface.

Significant transverse cracking of the LV 150-200 PEN., 4-6-SAND pavements did not initiate until the thermal regime of these pavements had reached a low temperature level comparable to that of the LV 150-200 PEN., 4-16-CLAY pavements at initiation of transverse cracking. It is shown in Figure 23 that significant transverse cracking of the LV 150-200 PEN., 4-6-SAND pavements occurred between

January 4 and 7, 1968 when the minimum daily temperatures ranged from -28° F to -32° F at the pavement surface, -25° F to -29° F at a depth of two inches below the surface and -16° F to -20° F at a depth of four inches below the surface. A considerable number of transverse cracks were observed to occur thereafter, in most cases coincident with low temperature periods of less intensity than observed during the first week of January, 1968.

The similarity in temperature conditions at corresponding locations within the asphaltic concretes of the 4-16-CLAY and 4-6-SAND pavements at the time of initiation of transverse cracking implied that whether a pavement does or does not crack is primarily dependent upon the low temperature properties of the asphaltic concrete. As the LV 150-200 PEN., 4-16-CLAY pavements exhibited only 40% of the frequency of Type 1 cracks observed in the LV 150-200 PEN., 4-6-SAND pavements, the frequency of transverse cracking appears to be dependent upon the low temperature properties of the asphaltic concrete and the degree of interaction between the asphaltic concrete and the underlying materials. This implies that the base course and subgrade offer a certain amount of restraint to the thermal contraction of the asphaltic concrete surface and thus affect the transverse cracking frequency of the pavement.

Similarly, transverse cracking in the LV 150-200 PEN., 10-0-CLAY pavement initiated during a low temperature period when the asphaltic concrete had cooled down to the low temperature levels shown in Figure 24. However, the transverse cracking frequency of this pavement was 40 percent of that observed in the 4-16-CLAY pavements, indicating that increased thickness of asphaltic concrete offers restraint to temperature induced contraction at the pavement surface and results in a reduction in the frequency of transverse cracking.

The first transverse cracking of the LV 300-400 PEN., 4-16-CLAY pavements, shown in Figure 25, initiated on January 4, 1968, after a prolonged period of low temperature duration and several days after the first cracking was observed in the LV 150-200 PEN., 4-16-CLAY pavements. The delay in initial cracking of the LV 300-400 PEN., 4-16-CLAY pavements is indicative of greater resistance to thermal cracking by asphaltic concretes of this asphalt grade than by concretes incorporating LV 150-200 PEN. grade asphalt.

Although the HV 150-200 PEN. and SC-5 asphalt pavements were subjected to the same low temperature gradients as the LV 150-200 PEN. and the LV 300-400 PEN. asphalt pavements, they have not exhibited any transverse cracking to date. More information is needed on the low temperature properties of all the materials incorporated in the test road pavements to ascertain the significant parameters which render pavements non-susceptible to transverse cracking.

An analysis of ten years of minimum daily temperature data was carried out to determine the frequency of recurrence of low temperature cycles in southern Manitoba. The coldest cycle was selected for each winter, without regard to the number of low temperature cycles of lesser intensity, and the intervals of recurrence established in a manner analogous to that used in hydrologic applications. In Figure 26 the coldest temperature period for any selected recurrence interval is the period with a minimum daily temperature intensity that will be equaled or exceeded on the average of one time in the selected interval.

The December 17, 1967 to January 11, 1968 low temperature cycle, during which most of the transverse cracking occurred, has been the most severe low temperature intensity period experienced during the first two years since the test road was constructed. It is shown in Figure 26 that the above low temperature period has an average recurrence interval ranging between 5 and 10 years, depending upon the temperature intensity selected. As the service life of a bituminous pavement, before major reconstruction is necessary, is generally in the order of 15 years, the occurrence of the above low temperature period is not considered abnormal for the climate in southern Manitoba. Correlation of the daily minimum ambient temperatures with the daily minimum asphaltic concrete temperatures for the above cycle indicated that a good correlation existed, as shown in Figures 27, 28 and 29. This permits estimation of pavement

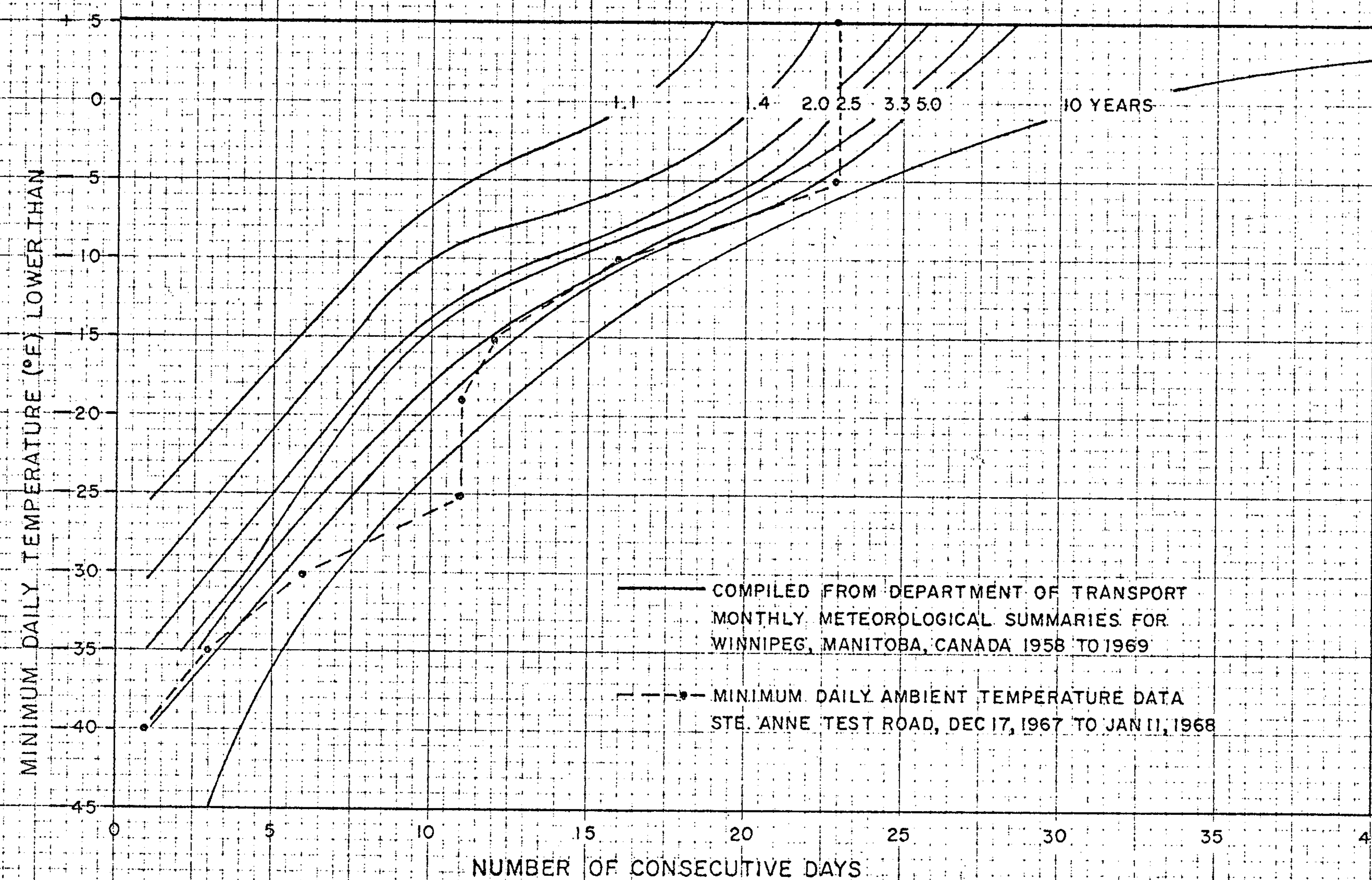


FIGURE 26. INTERVAL OF RECURRENCE AND DURATION OF THE LOWEST ANNUAL TEMPERATURE CYCLE

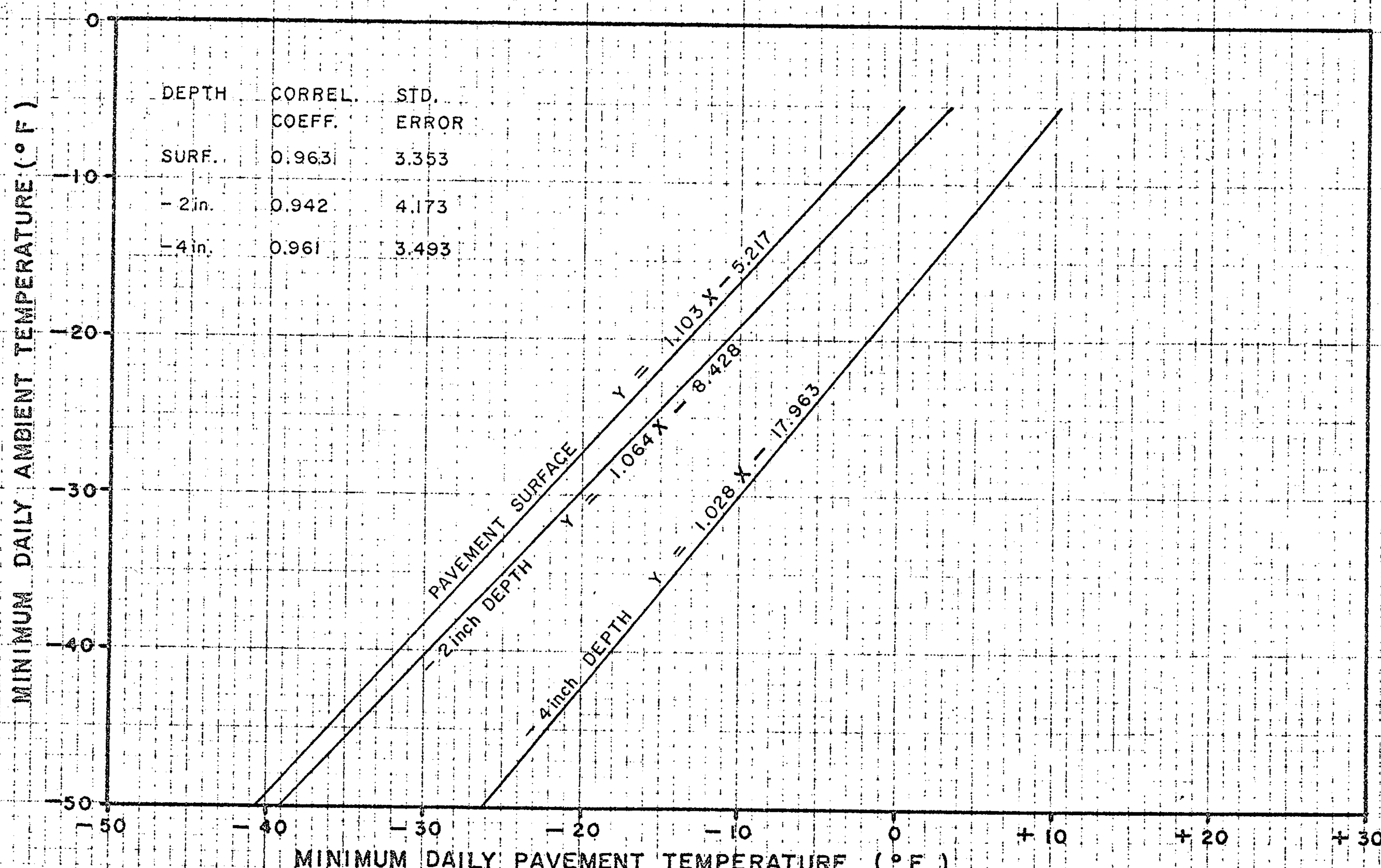


FIGURE 27. RELATIONSHIP BETWEEN AMBIENT AND PAVEMENT TEMPERATURES IN 4 – 16 CLAY PAVEMENTS DURING A LOW TEMPERATURE CYCLE

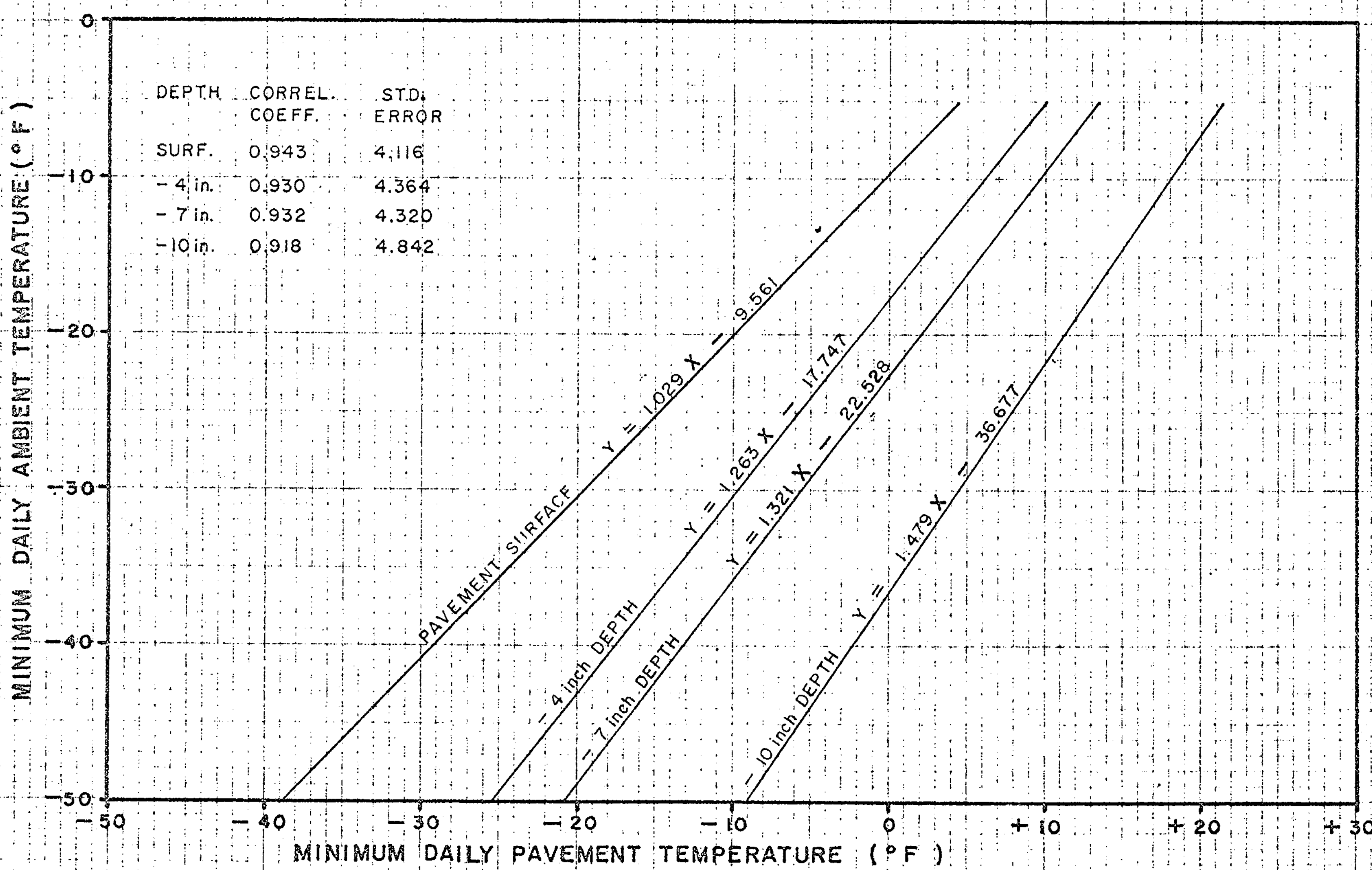


FIGURE 28. RELATIONSHIP BETWEEN AMBIENT AND PAVEMENT TEMPERATURES IN 10 - 0 - CLAY PAVEMENTS DURING A LOW TEMPERATURE CYCLE

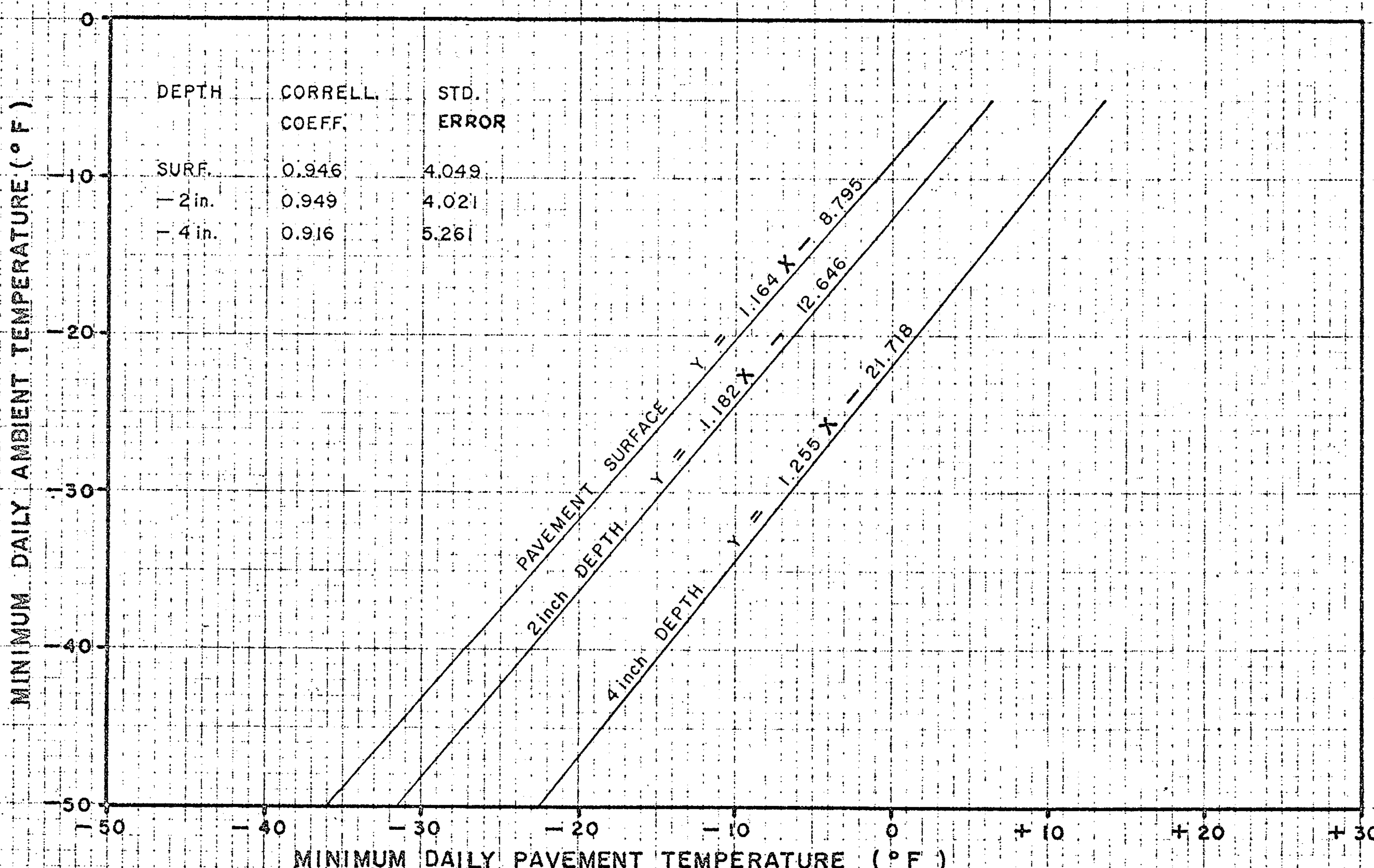


FIGURE 29. RELATIONSHIP BETWEEN AMBIENT AND PAVEMENT TEMPERATURES IN 4 — 6 — SAND PAVEMENTS DURING A LOW TEMPERATURE CYCLE

temperatures for similar temperature cycles from weather office data. Such information may be useful in estimating the magnitudes of thermally induced stresses in pavements in other parts of Manitoba.

Pavement Crack Studies

Inspection of the test road pavements indicated that the majority of the cracks were perpendicular to the direction of vehicular travel with the fractured faces oriented in a vertical plane, through the least cross-sectional area of the asphaltic concrete. This indicated that the transverse cracks resulted from the development of excessive tensile stresses within the pavement structure.

The pattern of transverse crack development in most pavements with clay subgrades followed a systematic process of cracking at long distances followed by subdivision of the long "slabs" into shorter segments on ensuing days until what is believed to be an equilibrium condition had been reached. This was interpreted as a form of relief from the stresses induced by thermal contraction of the pavement materials.

The pavements with sand subgrades did not crack in the above manner. The pattern of crack development appeared erratic. On numerous occasions the cracks in the LV 150-200 PEN., 4-6-SAND pavements appeared in clusters--a series of transverse cracks spaced four to ten feet apart. This manner of cracking may be attributed to the influence of the sand subgrade, the sand material exhibiting considerably lower tensile strength in a frozen condition than a clay subgrade in the same state.

The transverse cracks in the pavements with sand subgrades were different in nature from the cracks in the pavements with clay subgrades. Shortly after their occurrence, the transverse cracks in the LV 150-200 PEN. and LV 300-400 PEN., 4-16-CLAY pavements generally ranged in width from 1/32 to 3/16 inch at the pavement surface and extended either across the entire 24-foot pavement width or across an entire 12-foot lane. However, the transverse cracks in the LV 150-200 PEN., 4-6-SAND pavements were of minute width and difficult to trace across the pavement. Frequently they extended several feet across the pavement but did not extend to the pavement edge. While discernable at -30° F, many of the cracks were not visible when the temperature rose to -15° F. The differences in crack width exhibited by the clay and sand subgrades are believed to result from the differences in the properties of the subgrade materials and the nature of the interactions between the pavements and subgrades at low temperatures.

After approximately a year it was observed that all of the transverse cracks had increased in width a significant amount, as described on page 44. This phenomenon has been verified by crack width measurements in other investigations in Manitoba where it was found that not only was pavement crack width subject to daily and seasonal fluctuations, coincident with daily and seasonal thermal contraction of the "slabs", but also to annual increases in width up to a period of approximately three years. It is believed that the annual increases in crack width constitute a form of stress relief within the pavement.

The transverse crack detection circuits, situated at various depths in the pavements with clay subgrades, revealed that some transverse cracks initiated at the pavement surface and progressed downward through the asphaltic concrete. In a few instances the rate of propagation was extremely rapid and cracking through the asphaltic concrete was considered to be instantaneous. Judging from the great width of the cracks soon after initiation, instantaneous cracking was believed to be the prevalent form of crack propagation in the asphaltic concretes with clay subgrades.

The transverse crack detection circuits in the pavements with sand subgrades did not appear to be sufficiently sensitive to indicate accurately the initiation of the hairline transverse cracks, which were prevalent in these pavements. Pavement cores, taken across these cracks immediately after observation of crack occurrence, indicated that in some cases the asphaltic concrete had cracked only near the pavement surface. Neither the prevalence of the limited-depth cracks nor the rate of crack propagation down through the asphaltic concrete was assessed.

An x-ray photography technique was implemented to substantiate the indications of the transverse crack detection circuits and visual observations in regard to initiation of transverse cracking at the pavement surface and subsequent limited-depth crack propagation into the asphaltic concrete. The procedure used in the study was similar to that described by Slate and Olsefski (32) in a study of the internal structure and microcracking of portland cement concrete. Thin slices, approximately 0.5 cm. in thickness, were cut across transverse cracks in asphaltic concrete cores and irradiated. An industrial x-ray unit with a rating of 175 kilovolts (kv), and a fine-grained x-ray film were used. An exposure time of one minute at 165 kv and three milliamperes was used. A photograph of the asphaltic concrete slice is shown in Figure 30 with the arrow indicating the location of the transverse crack observed at the surface. In Figure 31 the transverse crack appears as a white line on the contact print of the x-ray image and is shown to propagate to a depth of approximately two inches, terminating in the proximity of the interface between the two lifts of asphaltic concrete.

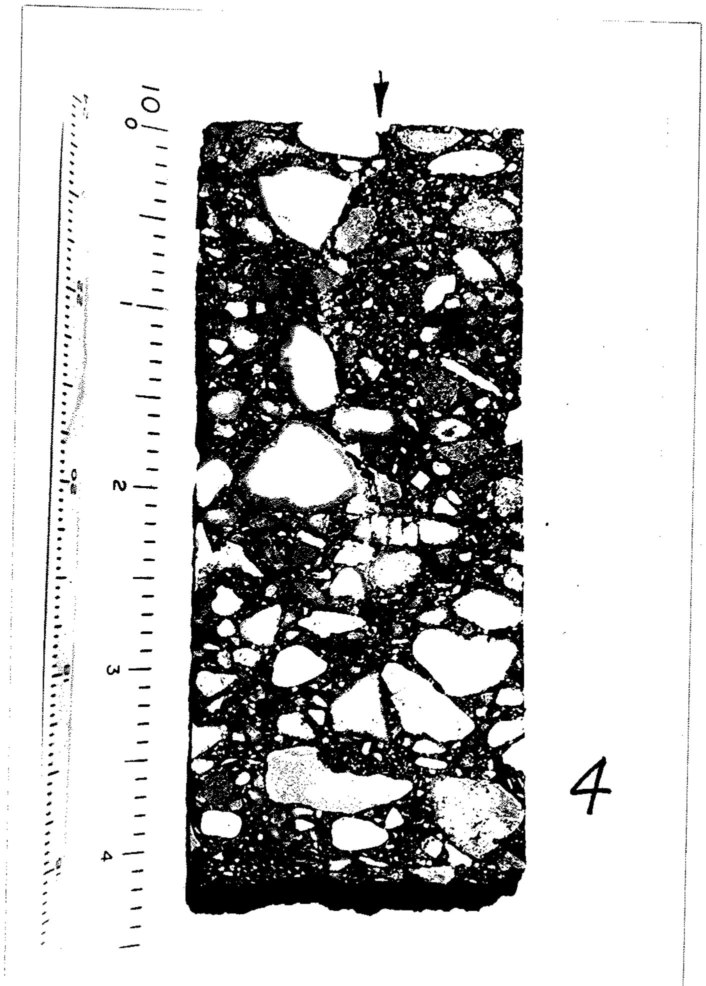


Figure 30. Photograph of asphaltic concrete slice with arrow indicating location of crack at surface

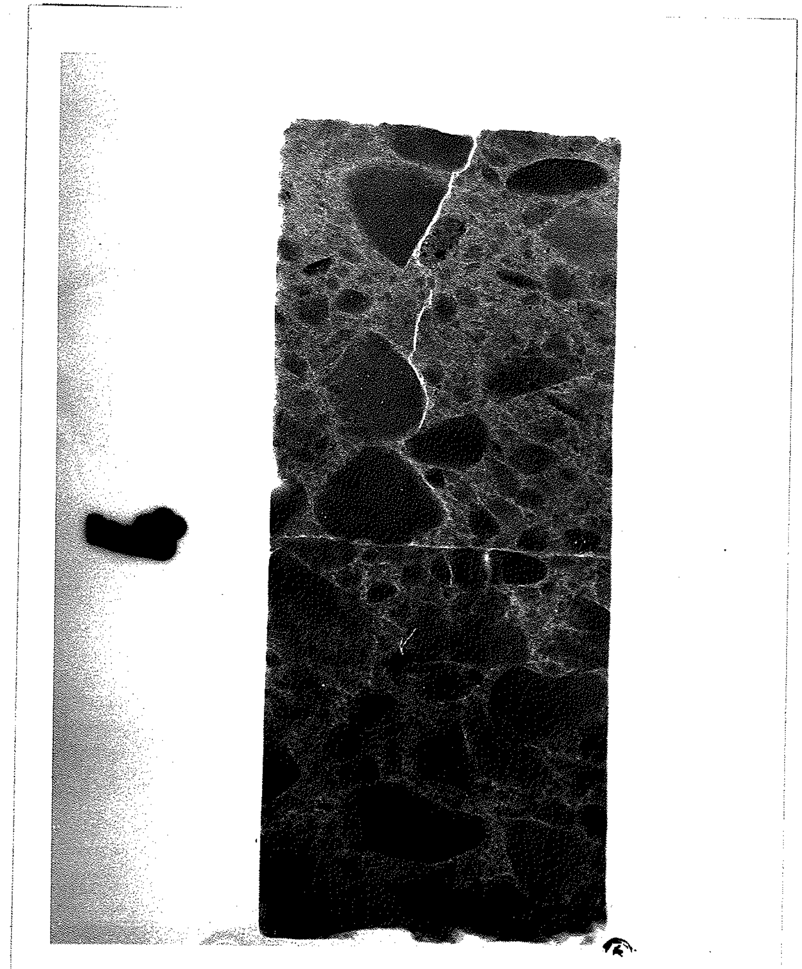


Figure 31. X-ray image of asphaltic concrete slice in Figure 30 with white line indicating depth of transverse crack propagation

CHAPTER VI

PRINCIPAL FINDINGS AND PROPOSED ADDITIONAL RESEARCH

Principal Findings

The results of the investigation indicate that the transverse cracking frequency of flexible pavements is affected by certain variables. The implications of the findings should be taken into consideration in pavement design to alleviate transverse pavement cracking. The statistical study of the interrelationships of transverse cracking frequencies among the Ste. Anne Test Road pavements served to isolate certain variables which had an effect on transverse pavement cracking.

1. There was no significant difference in transverse cracking frequencies between the traffic and the passing lanes indicating that traffic loading was insignificant in contributing to transverse cracking.
2. The asphalt content, in the range of one percent below Marshall optimum to one-half percent above Marshall optimum, was not significant in affecting pavement transverse cracking frequency.
3. The incorporation of portland cement filler,

although yielding significant improvement in Marshall design properties, was not significant in affecting transverse cracking frequency of pavements in the range of contents studied.

4. There was an interaction between the asphaltic concrete and the subgrade at low temperatures and the frequency of transverse pavement cracking was dependent upon the low temperature properties of both the asphaltic concrete and the subgrade.
5. Mixes incorporating high viscosity type asphaltic exhibited resistance to transverse cracking superior to that of mixes incorporating low viscosity type asphalts.
6. Mixes incorporating 'softer' grade (higher penetration) asphalts exhibited lower transverse cracking frequencies than mixes incorporating 'harder' grade (lower penetration) asphalts.
7. Asphalt was found to be the dominant variable affecting transverse pavement cracking. Transverse cracking of asphaltic concrete pavements can be alleviated through the selection of asphalts of the proper type and grade.
8. The frequency of transverse cracking was found to vary inversely to the thickness of the asphaltic concrete.

A study of the temperature data at the time of initiation of transverse cracking and investigations of the mode of asphaltic concrete fracture indicated the following.

1. Most of the transverse cracking occurred during a prolonged low temperature cycle during which the asphaltic concrete reached a low temperature level.
2. Transverse cracking initiated at the pavement surface and progressed downward through the asphaltic concrete.
3. Shortly after their occurrence, the transverse cracks in pavements with clay subgrades generally ranged in width from 1/32 to 3/16 inch at the pavement surface while the transverse cracks in pavements with sand subgrades were of minute width at the pavement surface.
4. In pavements with clay subgrades, the rate of crack propagation was generally rapid, on the borderline of being instantaneous. Several days after their occurrence, many of the transverse cracks in pavements with sand subgrades had propagated only to a limited depth into the asphaltic concrete surface and the rate of downward propagation could not be assessed.
5. A year after their occurrence, all of the transverse cracks had increased in width.
6. Transverse cracking is the result of thermally

induced stresses which exceed the ultimate tensile strength of the asphaltic concrete.

Recommended Additional Research

The Ste. Anne Test Road was designed to permit investigation of transverse pavement cracking and all aspects of pavement performance. The data presented in the thesis comprises only a part of the entire study. Further laboratory and field research is necessary to provide an objective basis for the development of specifications for materials and a balanced pavement design system, which will serve to alleviate transverse pavement cracking without introducing other forms of pavement distress mechanisms.

On the basis of observations to date, a laboratory evaluation of the low temperature properties of all the materials incorporated in the pavements is necessary to ascertain the significant parameters which render pavements susceptible to transverse cracking. Particular emphasis must be placed on the asphalt mixes in this regard and a method of predicting thermal fracture susceptibility should be evolved for bituminous mixes before they are placed in service. A number of existing methods requiring verification by field data were mentioned in Chapter II.

As asphalt type and grade appeared to be the dominant variables affecting transverse pavement cracking, the suitability of any asphalt designated for use in a specific environmental region must be evaluated on the basis of a minimum temperature intensity criterion indigenous to the region.

Aging of the asphalts in pavements must be taken into consideration and their aged properties should be related back to the original asphalt properties to provide a suitable basis for the specification of asphalts for extended periods of service.

All aspects of pavement distress must be investigated in the test road pavements with particular emphasis being placed on further development of pavement cracking and pavement surface distortion within the wheelpaths and at transverse cracks. The development of any undesirable performance characteristics should be related back to the mix properties and eliminated in future mix designs.

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APPENDIX I

QUALITY CONTROL DATA FOR THE STE. ANNE TEST ROAD PAVEMENTS

Table 4. Summary of Clay Subgrade Density Tests

SECTION NUMBER	MEAN DRY DENSITY (lb/cu.ft)	STANDARD DEVIATION	NUMBER OF TESTS	PERCENT OF STANDARD	MEAN MOISTURE (%)	STANDARD DEVIATION	NUMBER OF TESTS	PERCENT OF STANDARD
51	81.0	3.5	4	102	37.5	5.6	4	108
52	77.4	3.4	4	97	39.5	7.0	4	114
53	77.2	3.6	5	97	41.9	3.1	5	121
54	80.5	3.4	4	101	40.4	3.8	4	117
55	78.5	4.6	6	99	38.9	4.6	6	112
56	82.7	4.6	7	104	38.4	3.9	7	111
57	80.9	2.7	4	102	39.1	1.4	4	113
58	81.0	4.4	10	102	39.2	3.3	10	113
59	81.7	3.4	10	103	37.0	3.5	10	107
60	84.0	1.8	5	106	32.9	6.4	5	95
61	84.3	7.5	6	106	34.8	6.6	6	101
62	82.0	2.7	5	103	38.0	2.6	5	110
63	81.4	2.8	4	102	37.3	2.5	4	108
64 & 65	81.4	4.2	13	102	37.6	4.8	13	109

Standards by ASTM Test Method D 698-66T (Method A);
Dry Density = 79.6 lb./cu.ft.; Moisture content = 34.6%

Table 5. Summary of Base and Sub-Base Density Tests for Pavements with Clay Subgrades

SECTION NUMBER	"B" BASE		"A" BASE	
	Mean Dens. (pcf)	% of Std. Dens.	Mean Dens. (pcf)	% of Std. Dens.
51 s*	140.4 (1.3)	96	140.0	97
52 s	140.0 (0.9)	95		
53 s	140.2 (0.9)	95		
54 s	142.0 (1.0)	97	135.0	93
55 s	140.0 (0.8)	95	145.6	101
56 s	139.8 (0.6)	95.1	140.2	97
57 s	139.3 (0.9)	95		
58 s	139.6 (1.5)	95		
59 s	139.8 (0.9)	95	140.0	97
60 s	147.5 (0.8)	96	140.0	97
61 s	147.0	100	138.5	96
62 s	144.0	98		
63 s	145.2	99	141.5	98

*s: Standard Deviation of Test Data

Table 6. Summary of Subgrade and Base Density Tests for Pavements with Sand Subgrades

SECTION NUMBER	SAND SUBGR.		"A" BASE	
	Mean Dens. (pcf)	% of Std. Dens.	Mean Dens. (pcf)	% of Std. Dens.
66 s	122.0	96	142.0 (0.5)	98
67 s	124.5	98	143.1 (1.1)	99
68 s	126.0	99	140.5 (1.5)	97
69 s	134.2	105	142.9 (0.7)	99
70 s	127.5	100	142.9 (1.1)	99
71 s	135.0	106	140.6 (1.3)	97
72 s	133.3	105	139.8 (1.5)	97
73 s	129.8	102	139.6 (0.8)	97
74 s	125.8	99	141.4 (0.8)	98
75 s	134.8	106	140.2 (1.0)	97
76 s	131.8	104	138.8 (1.2)	96
77 s	126.5	99	142.9 (1.3)	99
78 s	133.6	105	144.4 (0.7)	100
79 s	134.7	106	143.2 (0.9)	99

Standards by ASTM Test Method D 698-66T (Method A) with N.W. McLeod's Stone Correction: "A" Dry Density = 144.4 pcf; "B" Dry Density = 147.0 pcf; Sand Subgrade Dry Density = 127.2 pcf.

Table 7. Summary of Base Course Aggregate Gradation Tests

SIEVE SIZE	GRADATION - Percent Passing				
	"B" BASE Pit 1	"B" BASE Pit 2	MAN. SPEC. B 3080R6	"A" BASE	MAN. SPEC. A 3080R6
1½"	100	100	100		
3/4"				100	100
5/8" s*				94.4 (1.1)	80 - 100
1/2" s	77.9 (2.3)	75.0 (2.9)			
#4 s	57.6 (2.3)	52.8 (3.6)	35-75	60.3 (4.1)	40-70
#10 s	39.5 (2.9)	34.6 (5.2)	25-65	39.8 (5.0)	25-55
#40 s	17.0 (2.2)	18.5 (5.5)	15-35	18.4 (3.0)	15-30
#200 s	4.8 (1.0)	10.4 (3.2)	4-18	11.1 (1.9)	4-15
Percent Crush	42.0 (4.1)		25% Min.	50.8 (6.7)	35% Min.
NO TESTS	24	21		55	

*s: Standard Deviation of Test Data

Table 8. Summary of Bituminous Mix Aggregate Gradation Tests

SIEVE SIZE	MEAN GRADATION - PERCENT PASSING			SPECIFICATIONS	
	LIMESTONE AGGREGATE	LIMESTONE WITH FILLER	IGNEOUS WITH FILLER	MANITOBA 3060 "A" Spec.	SWEDISH AB 16T Spec.
3/4" s*	100.0	100.0	100.0	100	100
5/8" s	100.0	100.0	99.9	100	85 - 100
3/8" s	81.1 (2.6)	81.7 (2.7)	73.9 (5.5)	70 - 85	65 - 85
#4 s	56.6 (3.2)	58.1 (3.1)	48.8 (2.0)	50 - 65	45 - 63
#10 s	43.3 (3.1)	45.0 (3.0)	33.6 (2.9)	35 - 50	26 - 42
#20 s	31.3 (3.0)	33.3 (2.6)	23.9 (1.9)		17 - 31
#40 s	19.8 (3.1)	22.4 (2.6)	16.0 (0.9)	15 - 30	12 - 26
#80 s	8.0 (1.1)	10.9 (0.9)	9.2 (0.7)		7 - 18
#200 s	2.9 (0.6)	5.6 (0.8)	5.5 (0.9)	2 - 6	5 - 10
NO. TESTS	245	108	29		

*s: Standard Deviation of Test Data.

Table 9. SUMMARY OF ASPHALT PROPERTIES: STE. ANNE TEST ROAD, MANITOBA

Asphalt Type	As Supplied						After T.F.O.T. (d)						
	Penetration @ (a)			η @ (b)	S.P.	Tg'		Penetration @			η @	S.P.	Tg'
	77	39.2	32.0	140°F	°F (c)	°F (e)	% Loss	77	39.2	32.0	140 °F	°F	°F
150/200 (LV)	200.5 203.0	39.7	20.2	187.3	114.1	-5.2	0.24	86.0	23.1	18.6	549.3	124.3	+4.7
150/200 (HV)	171.2 172.0	59.5	34.7	485.9	106.8	-13.5	0.27	83.6	34.1	21.5	1628.9	119.2	-18.6
300/400 Pen.	367.2	75.7	43.1	83.1	104.7	+2.2	0.03	172.0	45.3	28.8	198.7	119.0	-6.5
SC-5	--	275.6	161.1	84.3	91.3	-15.5	5.70	76.7	32.0	25.3	1364.4	123.0	-13.6

(a) ASTM D-5-65
(b) " D-2171-66
(c) " D-36-66T
(d) " D-1754-66T
(e) Schmidt and Santucci: Proc. AAPT. Vol. 35, 1966, pp. 61-90.

Asphalt Type	As Supplied Viscosity (f)												Thin Film Oven Test Residue Viscosity											
	η (77°F) poise x 10^5						η (39.2°F) poise x 10^8						η (77°F) poise x 10^5						η (39.2°F) poise x 10^8					
	Shear Rate, sec^{-1}						Shear Rate, sec^{-1}						Shear Rate, sec^{-1}						Shear Rate, sec^{-1}					
	10^{-3}	10^{-2}	10^{-1}	10^{0}	FS*	"c"	10^{-3}	10^{-2}	10^{-1}	10^{0}	FS*	"c"	10^{-3}	10^{-2}	10^{-1}	10^{0}	FS*	"c"	10^{-3}	10^{-2}	10^{-1}	10^{0}	FS*	"c"
150/200 (LV)	2.39	2.19	2.02	1.84	2.03	0.973	3.30	2.18	1.45	0.94	3.12	0.824	30.0	18.6	11.5	7.0	15.4	0.792	15.70	6.60	2.71	1.15	17.10	0.625
150/200 (HV)	3.88	3.25	2.71	2.26	2.84	0.967	1.05	1.05	1.05	1.05	1.05	0.999	16.8	15.9	15.2	14.2	15.8	0.996	7.01	6.20	5.60	4.95	6.82	0.970
300/400	1.61	1.32	1.09	0.89	1.09	0.894	2.11	0.75	0.27	0.09	1.34	0.549	6.0	4.4	3.2	2.2	3.4	0.876	3.65	1.58	0.68	0.29	2.93	0.626
SC-5	1.21	0.80	0.52	0.34	0.49	0.806	0.14	0.09	0.06	0.04	0.09	0.798	28.3	22.8	18.0	14.2	20.9	0.886	8.40	4.52	2.42	1.31	8.21	0.737

(f) "Proposed Method of Test for Viscosity of Asphalt with a Sliding Plate Viscometer at Controlled Rates of Shear". ASTM, 1968 Standards: Part II, page 871.

Power input = 1000 ergs.

Research Council of Alberta - 1968

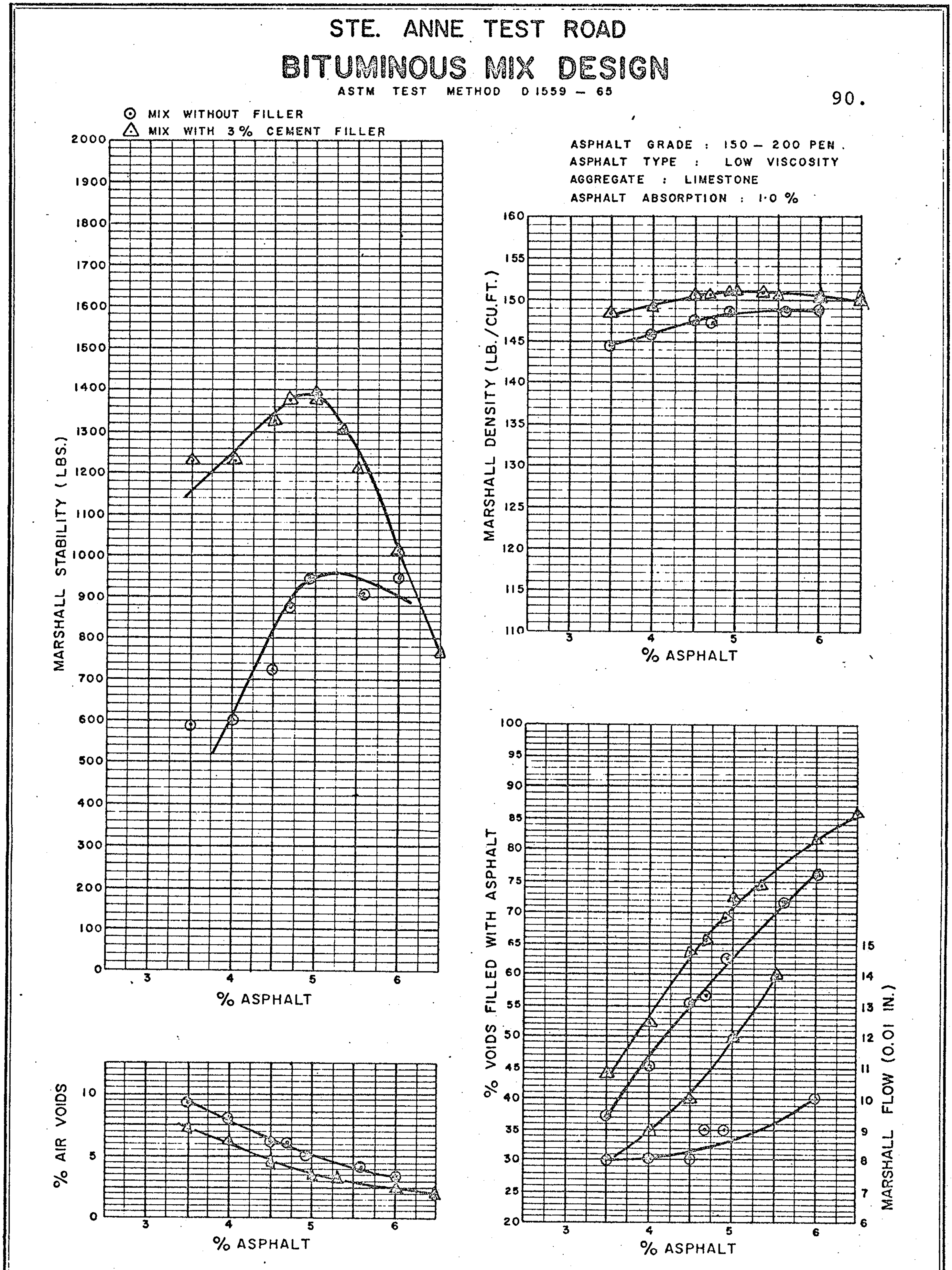

Figure 32. Design of LV 150-200 PEN. Asphalt Mixes

STE. ANNE TEST ROAD

BITUMINOUS MIX DESIGN

ASTM TEST METHOD D 1559 — 65

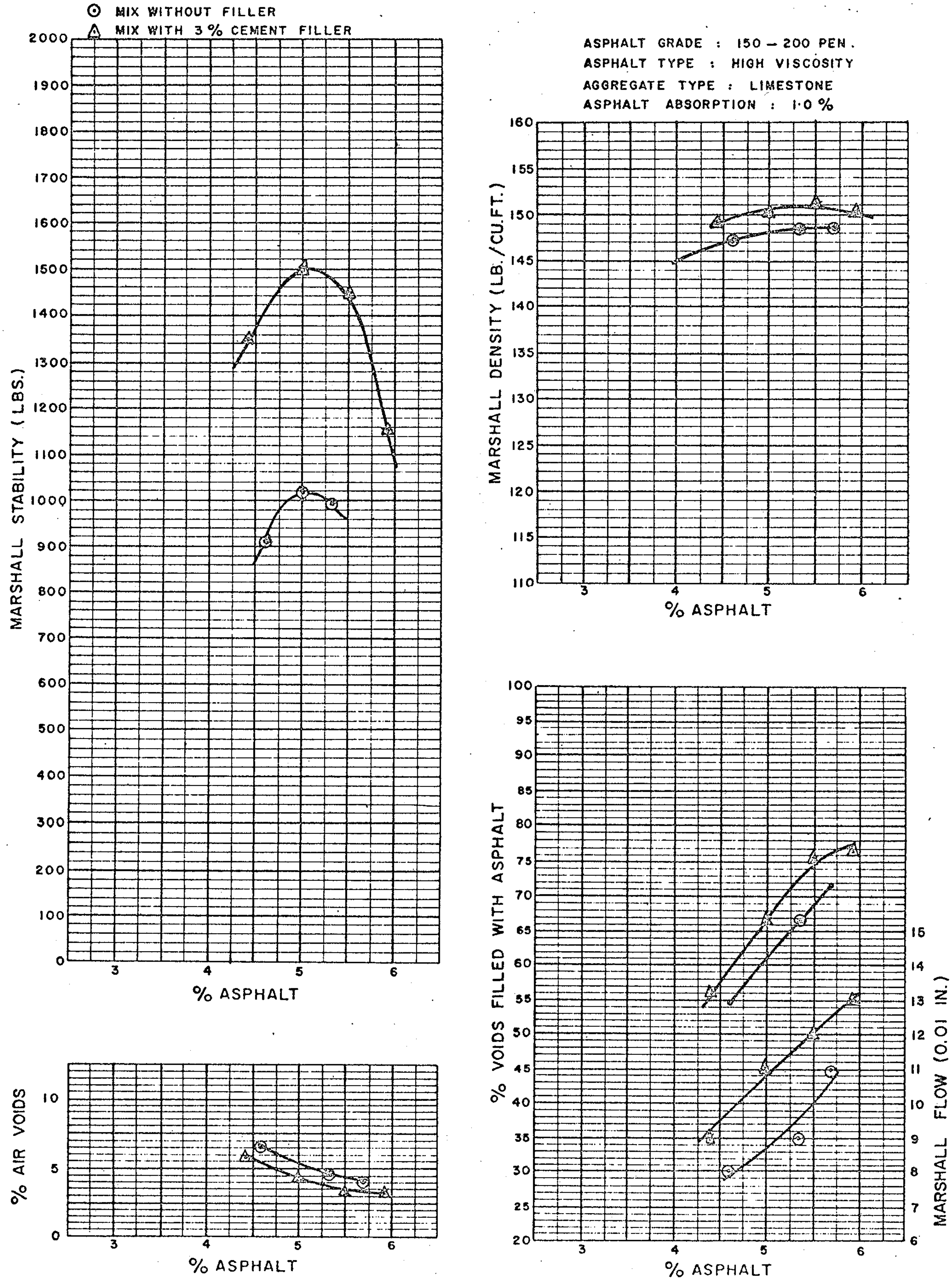

Figure 33. Design of HV 150-200 PEN. Asphalt Mixes

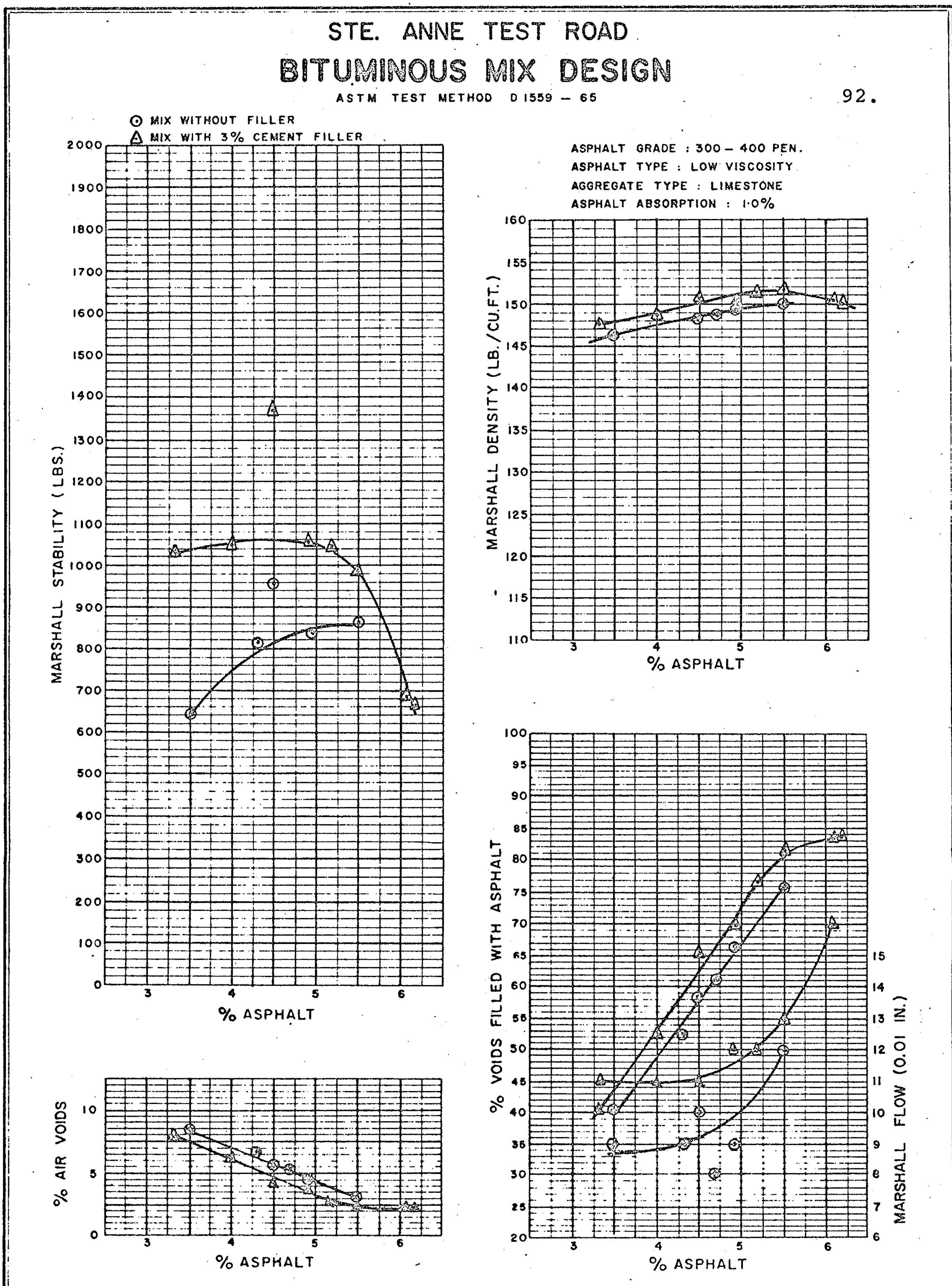

Figure 34. Design of LV 300-400 PEN. Asphalt Mixes

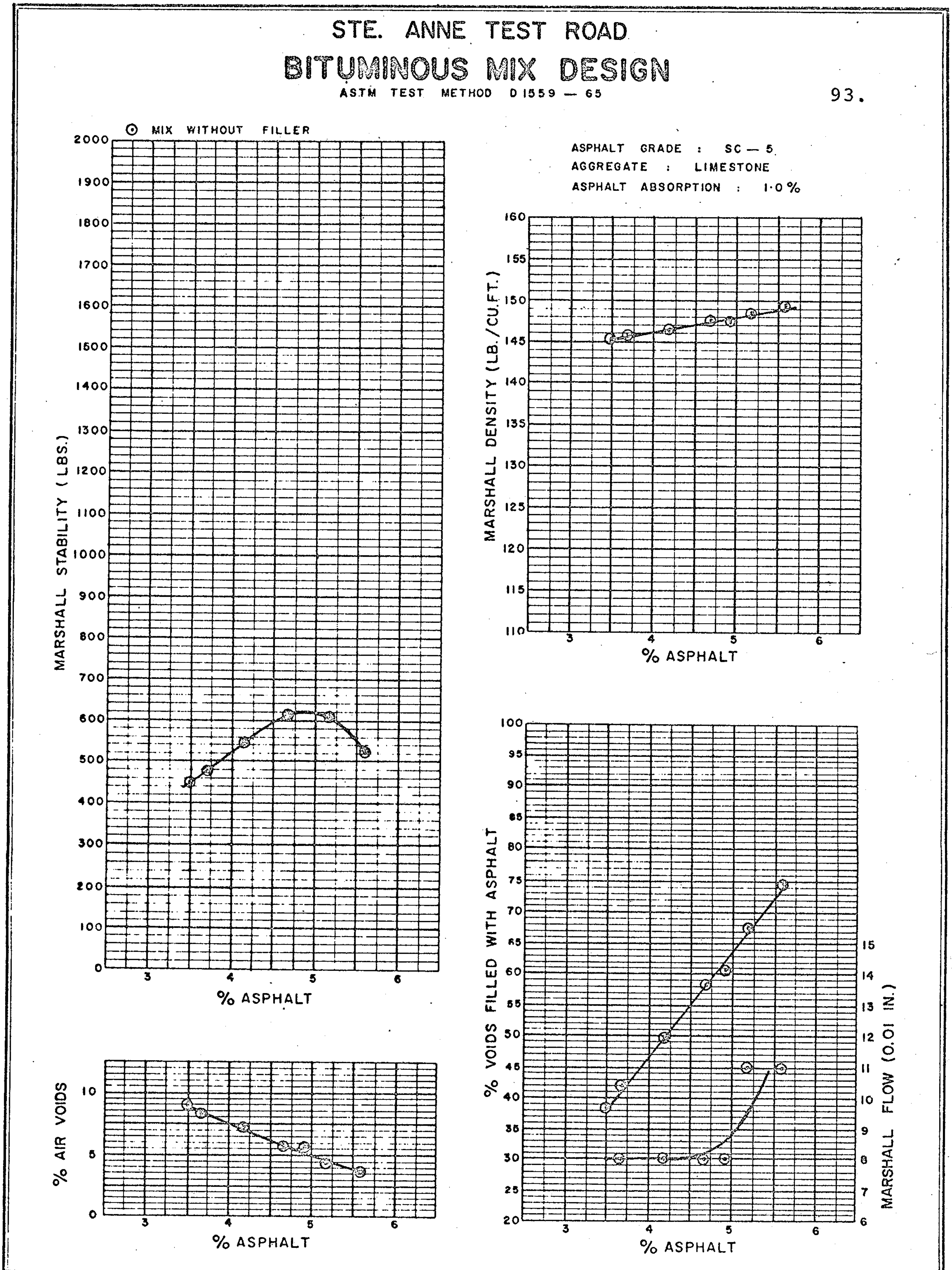

Figure 35. Design of SC-5 Asphalt Mixes

Table 10. Design Properties of Bituminous Mixes

ASPHALT	FILLER	MIX TYPE	ASPHALT CONTENT (%)	MARSH. STAB. (Lb.)	MARSH. FLOW .01 in.	MARSH. DENS. (pcf)	EFF. AIR (%)	VOIDS FILLED (%)
LV 150-200 PEN		Below Opt.	4.0	610	8	146.0	8.0	46.5
		Optimum	5.0	950	9	148.4	5.0	63.0
		Above Opt.	5.5	950	9	148.7	4.2	70.0
	X	Below Opt.	4.0	1250	9	149.5	6.5	53.0
	X	Optimum	5.0	1390	12	151.0	3.5	71.0
HV 150-200 PEN		Below Opt.	4.0			145.0		
		Optimum	5.0	990	9	148.0	5.4	61.0
	X	Optimum	5.0	1500	11	150.5	4.3	66.5
	X	Igneous	5.0	1280	12	155.1	2.6	81.7
LV 300-400 PEN		Below Opt.	4.0	750	9	147.5	7.0	48.5
		Optimum	5.0	850	10	149.4	4.0	66.5
	X	Below Opt.	4.0	1050	11	149.0	6.2	53.5
	X	Optimum	5.0	1030	12	151.0	3.3	72.0
HV SC-5		Optimum	5.0	620	9	148.0	5.0	64.0

Table 11. Summary of Asphalt and -#200 Mesh Material Content Control

Four-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	4-16-CLAY STRUCTURE			4-6-SAND STRUCTURE		
			SECTION NUMBER	PERCENT ASPHALT	PERCENT -#200	SECTION NUMBER	PERCENT ASPHALT	PERCENT -#200
LV 150-200 PEN		Below Opt. s	54	4.1 (0.4)	3.0 (0.6)	74	4.2 (0.4)	3.0 (1.1)
		Optimum s	63	4.8 (0.3)	3.0 (0.8)	67	4.7 (0.4)	3.1 (0.7)
		Above Opt. s	- -	- -	- -	77	5.5 (0.3)	6.4 (0.9)
	X	Below Opt. s	55	4.2 (0.3)	6.1 (0.9)	76	4.2 (0.2)	6.1 (0.5)
	X	Optimum s	53	5.0 (0.3)	5.3 (1.0)	75	5.2 (0.2)	6.0 (0.9)
HV 150-200 PEN		Below Opt. s	57	4.3 (0.3)	2.8 (0.6)	73	4.0 (0.3)	2.7 (0.3)
		Optimum s	62	5.1 (0.1)	2.7 (0.4)	66	5.2 (0.4)	2.6 (0.4)
	X	Optimum s	56	5.4 (0.1)	4.6 (1.1)	72	5.5 (0.1)	5.3 (0.2)
	X	Igneous s	51	5.4 (0.3)	5.7 (0.8)	78	5.1 (0.3)	5.7 (0.4)
LV 300-400 PEN		Below Opt. s	58	3.9 (0.3)	3.0 (0.6)	71	3.6 (0.2)	3.1 (0.6)
		Optimum s	61	4.8 (0.3)	2.9 (0.4)	68	4.7 (0.3)	2.7 (0.7)
	X	Below Opt. s	59	4.2 (0.1)	5.7 (0.8)	70	4.1 (0.2)	5.8 (0.7)
	X	Optimum s	60	5.3 (0.3)	5.6 (0.6)	69	5.0 (0.3)	5.8 (0.9)
HV SC-5		Optimum s	52	4.9 (0.3)	3.0 (0.5)	79	4.8 (0.4)	3.0 (0.6)

Ten-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	10-0-CLAY STRUCTURE		
			SECTION NUMBER	PERCENT ASPHALT	PERCENT -#200
LV 150-200 PEN		Optimum s	64	4.8 (0.3)	2.8 (0.7)
HV 150-200 PEN		Optimum s	65	5.1 (0.4)	2.6 (0.4)

Table 12. Pavement Densities of Traffic Lane (Lane-Wells Road Logger Readings October 25, 1967)

Four-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	4-16-CLAY STRUCTURE			4-6-SAND STRUCTURE		
			SECTION NUMBER	DENSITY (PCF)	% OF STANDARD	SECTION NUMBER	DENSITY (PCF)	% OF STANDARD
LV 150-200 PEN		Below Opt. s	54	145.6 (0.7)	100	74	145.2 (1.2)	99
		Optimum s	63	147.2 (0.7)	99	67	146.4 (1.0)	99
		Above Opt. s	--	--	--	77	149.2 (0.7)	100
	X	Below Opt. s	55	146.5 (0.7)	98	76	145.7 (1.2)	97
	X	Optimum s	53	147.6 (0.9)	98	75	148.9 (0.6)	99
HV 150-200 PEN		Below Opt. s	57	145.1 (0.8)	100	73	145.5 (0.8)	100
		Optimum s	62	146.6 (0.8)	99	66	147.8 (1.0)	100
	X	Optimum s	56	150.0 (0.9)	100	72	150.2 (0.8)	100
	X	Igneous s	51	150.8 (1.2)	97	78	151.5 (1.3)	98
LV 300-400 PEN		Below Opt. s	58	143.7 (1.1)	97	71	144.4 (0.9)	98
	X	Optimum s	61	145.1 (1.0)	97	68	146.8 (0.9)	98
	X	Below Opt. s	59	145.5 (1.0)	98	70	145.7 (1.0)	98
		Optimum s	60	147.6 (1.2)	98	69	147.9 (0.8)	98
HV SC-5		Optimum s	52	145.9 (1.1)	99	79	146.5 (0.6)	99

Ten-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	10-0-CLAY STRUCTURE		
			SECTION NUMBER	DENSITY (PCF)	% OF STANDARD
LV 150-200 PEN		Optimum s	64	147.1 (0.9)	99
HV 150-200 PEN		Optimum s	65	147.8 (0.8)	100

Table 13. Pavement Densities of Passing Lane (Lane-Wells Road Logger Readings October 25, 1967)

Four-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	4-16-CLAY STRUCTURE SECTION NUMBER	4-16-CLAY STRUCTURE DENSITY (PCF)	4-16-CLAY STRUCTURE % OF STANDARD	4-6-SAND STRUCTURE SECTION NUMBER	4-6-SAND STRUCTURE DENSITY (PCF)	4-6-SAND STRUCTURE % OF STANDARD
LV 150-200 PEN		Below Opt. s	54	143.2 (0.9)	98	74	145.1 (1.0)	99
		Optimum s	63	147.0 (0.7)	99	67	148.0 (0.8)	100
		Above Opt. s	—	—	—	77	148.9 (1.1)	100
	X	Below Opt. s	55	144.6 (1.1)	97	76	145.6 (0.8)	97
	X	Optimum s	53	147.6 (0.8)	98	75	148.9 (1.2)	99
HV 150-200 PEN		Below Opt. s	57	143.6 (1.0)	99	73	144.6 (0.9)	100
		Optimum s	62	148.1 (0.7)	100	66	148.2 (0.9)	100
	X	Optimum s	56	150.4 (0.8)	100	72	150.3 (1.2)	100
	X	Igneous s	51	150.3 (1.3)	97	78	151.3 (1.4)	98
LV 300-400 PEN		Below Opt. s	58	142.4 (0.8)	97	71	144.6 (0.9)	98
		Optimum s	61	146.7 (1.1)	98	68	146.2 (0.8)	98
	X	Below Opt. s	59	143.1 (1.3)	96	70	145.0 (0.9)	97
	X	Optimum s	60	146.9 (1.1)	97	69	148.2 (0.8)	98
HV SC-5		Optimum s	52	146.3 (0.7)	99	79	147.5 (0.7)	100

Ten-Inch Asphaltic Concrete Sections:

ASPHALT	FILLER	MIX TYPE	10-0-CLAY STRUCTURE SECTION NUMBER	10-0-CLAY STRUCTURE DENSITY (PCF)	10-0-CLAY STRUCTURE % OF STANDARD
LV 150-200 PEN		Optimum s	64	147.3 (0.6)	99
HV 150-200 PEN		Optimum s	65	147.5 (0.8)	100

Table 14. 1968 Peak Mean Benkelman Beam Rebound Readings

4-16-CLAY Sections:

SEC. NO.	DATE	REBOUND (0.001 in.)		PAVE. TEMP. °F
		MEAN	s*	
51	June 17	55	3	87
52	June 17	54	2	87
53	June 17	53	2	87
54	June 17	53	2	87
55	June 17	51	2	87
56	June 17	46	3	87
57	June 17	46	2	87
58	May 30	42	4	66
59	June 17	44	3	87
60	June 17	39	3	87
61	June 17	44	3	87
62	June 17	39	3	87
63	May 30	46	7	66

4-6-SAND Sections:

SEC. NO.	DATE	REBOUND (0.001 in.)		PAVE. TEMP. °F
		MEAN	s*	
66	June 11	17	3	59
67	June 17	17	3	79
68	May 30	17	2	51
69	May 30	16	2	51
70	June 11	14	3	59
71	June 11	17	3	59
72	June 11	17	1	59
73	June 11	14	4	59
74	June 11	14	3	59
75	June 17	14	2	79
76	June 17	12	0	79
77	June 11	13	3	59
78	June 11	14	3	59
79	June 17	12	2	79

*s: Standard Deviation of 6 Readings per Section.

10-0-CLAY Sections:

SEC. NO.	DATE	REBOUND (0.001 in.)		PAVE. TEMP. °F
		MEAN	s*	
64	June 17	47	2	83
65	June 17	47	1	83

APPENDIX II

STATISTICAL ANALYSES OF STE. ANNE TEST ROAD DATA

Statistical Comparison of Asphalt Contents

The Student's t-test was used to determine the significance level of the difference between the mean asphalt contents of the below optimum asphalt content mixes and the optimum asphalt content mixes using the formula*:

$$t = \frac{\bar{X}_1 - \bar{X}_2}{\sqrt{\frac{(N_1 - 1)\, s_1^2 + (N_2 - 1)\, s_2^2}{N_1 + N_2 - 2} \cdot \left(\frac{1}{N_1} + \frac{1}{N_2}\right)}}$$

where:

t = number of standard deviations from the mean for a given number of degrees of freedom,

$\bar{X}_1, \bar{X}_2$ = means of the samples,

N_1, N_2 = sizes of the samples,

s_1, s_2 = standard deviations of the samples.

Comparisons were made between sections at significance levels of ten, five and one percent, using the data in Table 11.

The summary, tabulated in Table 15 shows that for paired sections of the same asphalt type and grade and aggregate gradation, there is a significant difference between the means of the asphalt contents at the one percent level.

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*Hoel, P. G., "Elementary Statistics", John Wiley & Sons, Inc., 1967, pp. 177.

This indicates that although the below optimum and optimum asphalt content mixes have similar probability distributions, there is a distinct difference in the mean asphalt content between mixes.

Table 15. Significance Levels of the Differences Between Mean Asphalt Contents

Below Optimum Asphalt Content Section No.	54	55	57	58	59	74	67	76	73	71	70
Optimum Asphalt Content Section No.	63	53	62	61	60	67	77	75	66	68	69
Significance Level	1%	1%	1%	1%	1%	1%	1%	1%	1%	1%	1%

Statistical Comparison of Mineral Filler Contents

The total percent of material finer than the #200 mesh sieve was selected as the criteria indicative of the degree of control attained in the addition of the portland cement filler. The t-test was used in the manner described previously to determine the significance level of the differences between the mean cement filler contents of the no-filler mixes and the filler mixes. The data used appears in Table 11.

The summary, tabulated in Table 16 shows that for paired sections of the same asphalt type and grade and for mixes of similar asphalt content, there is a significant difference between the means of the filler contents at the one percent level. This indicates that although the no-filler and filler mixes have similar probability distributions, there is a distinct difference in the mean minus #200 mesh material content between the mixes.

Table 16. Significance Levels of the Differences Between Mean Percents Finer than the No. 200 Mesh

No-Filler Mix Section No.	54	63	62	58	61	74	67	66	71	68
Mix With Filler Section No.	55	53	56	59	60	76	75	72	70	69
Significance Level	1%	1%	1%	1%	1%	1%	1%	1%	1%	1%

<u>Comparison of Transverse Cracking Between the Traffic Lanes and the Passing Lanes</u>

A comparison of transverse cracking between the lanes was carried out for each pavement section. Separate analyses were conducted using Type 1 crack data and data for all of the crack types combined. The t-test was implemented using the transverse crack data appearing in Tables 17 to 24.

Table 25 shows the levels at which the differences between the means were not significant for the sections that exhibited transverse cracking. With the exception of a few cases, the statistical analyses indicate that the difference

TABLE 17

SUMMARY OF TYPE I CRACK SPACING
TRAFFIC LANE
CLAY SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x}$ = 21 ft. s = 8.2 N = 16	$\bar{x}$ = 18 ft. s = 6.7 N = 20	—	$\bar{x}$ = 19.5 ft.
				CEMENT FILLER	$\bar{x}$ = 18 ft. s = 9.2 N = 20	$\bar{x}$ = 20 ft. s = 8.8 N = 17	—	$\bar{x}$ = 19 ft.
				COLUMN MEANS	$\bar{x}$ = 19.5 ft.	$\bar{x}$ = 19 ft.		Grand Mean $\bar{x}$ = 19 ft.
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x}$ = 46 ft. s = 24.5 ft. N = 7	—	$\bar{x}$ = 46 ft.
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				CEMENT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				COLUMN MEANS	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$		Grand Mean $\bar{x} \rightarrow \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
		10 IN	NONE	WITHOUT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
300-400 PEN.	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x}$ = 45 ft. s = 6.5 N = 8	$\bar{x}$ = 37 ft. s = 10.4 N = 8	—	
				CEMENT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x}$ = 46 ft. s = 30.3 N = 6	—	
				COLUMN MEANS				
SC-5				WITHOUT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$

$\bar{x}$ = Mean Crack Spacing   s = Standard Deviation
N = Sample Size (No. Cracks − 1)

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TABLE 18

SUMMARY OF TYPE I CRACK SPACING
PASSING LANE
CLAY SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x}$ = 21 ft. S = 8.0 N = 16	$\bar{x}$ = 19 ft. S = 6.5 N = 19	—	$\bar{x}$ = 20 ft.
				CEMENT FILLER	x = 21 ft. S = 9.0 N = 17	$\bar{x}$ = 25 ft. S = 7.7 N = 14	—	$\bar{x}$ = 23 ft.
				COLUMN MEANS	$\bar{x}$ = 21 ft.	$\bar{x}$ = 22 ft.		Grand Mean $\bar{x}$ = 21 ft.
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x}$ = 53 ft. S = 18.4 N = 7	—	$\bar{x}$ = 53 ft.
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} \to \infty$	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				CEMENT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				COLUMN MEANS	$\bar{x} \to \infty$	$\bar{x} \to \infty$		Grand Mean $\bar{x} \to \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
300-400 PEN.	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x}$ = 121 ft. S = 80 N = 3	$\bar{x}$ = 37 S = 10.4 N = 8	—	
				CEMENT FILLER	$\bar{x} \to \infty$	$\bar{x}$ = 43 ft. S = 25.8 N = 7	—	
				COLUMN MEANS				
SC-5				WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$

$\bar{x}$ = *Mean Crack Spacing* $\quad$ *s = Standard Deviation*
N = Sample Size (No. Cracks − 1)

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TABLE 19

SUMMARY OF TYPE I CRACK SPACING

TRAFFIC LANE

SAND SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}$ = 7 ft. S = 5.7 N = 51	$\bar{x}$ = 6 ft. S = 3.7 N = 61	$\bar{x}$ = 8 ft. S = 5.8 N = 45	$\bar{x}$ = 7 ft.
				CEMENT FILLER	$\bar{x}$ = 9 ft. S = 7.1 N = 36	$\bar{x}$ = 9 ft. S = 5.3 N = 39	—	$\bar{x}$ = 9 ft.
				COLUMN MEANS	$\bar{x}$ = 8 ft.	$\bar{x}$ = 7.5 ft.		*Grand Mean* $\bar{x}$ = 8 ft.
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				CEMENT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				COLUMN MEANS	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$		*Grand Mean* $\bar{x} \rightarrow \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
300-400 PEN.	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				CEMENT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				COLUMN MEANS	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$		*Grand Mean* $\bar{x} \rightarrow \infty$
SC-5				WITHOUT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$

$\bar{x}$ = *Mean Crack Spacing* s = *Standard Deviation*
N = *Sample Size (No. Cracks − 1)*

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TABLE 20

SUMMARY OF TYPE I CRACK SPACING

PASSING LANE

SAND SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}$ = 8 ft. s = 6.4 N = 47	$\bar{x}$ = 4 ft. s = 3.5 N = 75	$\bar{x}$ = 9 ft. s = 7 N = 37	$\bar{x}$ = 7 ft.
				CEMENT FILLER	$\bar{x}$ = 13 ft. s = 14.7 N = 26	$\bar{x}$ = 8 ft. s = 3.9 N = 46	—	$\bar{x}$ = 10.5 ft.
				COLUMN MEANS	$\bar{x}$ = 10.5 ft.	$\bar{x}$ = 6 ft.	$\bar{x}$ = 9 ft.	Grand Mean $\bar{x}$ = 8 ft.
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$		$\bar{x} \rightarrow \infty$
				CEMENT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				COLUMN MEANS	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	Grand Mean $\bar{x} \rightarrow \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
300-400 PEN.	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				CEMENT FILLER	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$
				COLUMN MEANS	$\bar{x} \rightarrow \infty$	$\bar{x} \rightarrow \infty$	—	Grand Mean $\bar{x} \rightarrow \infty$
SC-5				WITHOUT FILLER	—	$\bar{x} \rightarrow \infty$	—	$\bar{x} \rightarrow \infty$

$\bar{x}$ = Mean Crack Spacing s = Standard Deviation
N = Sample Size (No. Cracks −1)

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TABLE 21

SUMMARY OF ALL TYPE CRACK SPACING

TRAFFIC LANE

CLAY SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} = 19$ $s = 9.0$ $N = 18$	$\bar{x} = 16$ $s = 5.9$ $N = 23$	—	$\bar{x} = 17.5$ ft.
				CEMENT FILLER	$\bar{x} = 16$ $s = 9.7$ $N = 22$	$\bar{x} = 20$ $s = 8.8$ $N = 17$	—	$\bar{x} = 18.0$ ft.
				COLUMN MEANS	$\bar{x} = 17.5$ ft.	$\bar{x} = 18.0$ ft.		Grand Mean $\bar{x} = 18.0$ ft.
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x} = 46$ $s = 24.5$ $N = 7$	—	$\bar{x} = 46$ ft.
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} \to \infty$	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				CEMENT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				COLUMN MEANS	$\bar{x} \to \infty$	$\bar{x} \to \infty$		Grand Mean $\bar{x} \to \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
300-400 PEN.	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} = 121$ $s = 80$ $N = 3$	$\bar{x} = 37$ $s = 10.4$ $N = 8$	—	
				CEMENT FILLER	$\bar{x} \to \infty$	$\bar{x} = 43$ $s = 25.8$ $N = 7$	—	
				COLUMN MEANS				
SC-5				WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$

$\bar{x}$ = Mean Crack Spacing s = Standard Deviation
N = Sample Size (No. Cracks − 1)

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TABLE 22

SUMMARY OF ALL TYPE CRACK SPACING

PASSING LANE

CLAY SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} = 20$ $s = 8.4$ $N = 17$	$\bar{x} = 19$ $s = 6.5$ $N = 19$	—	$\bar{x} = 19.5$ ft.
				CEMENT FILLER	$\bar{x} = 21$ $s = 9.0$ $N = 17$	$\bar{x} = 25$ $s = 7.7$ $N = 14$	—	$\bar{x} = 23.0$ ft.
				COLUMN MEANS	$\bar{x} = 20.5$ ft.	$\bar{x} = 22.0$ ft.		*Grand Mean* $\bar{x} = 21$ ft.
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x} = 53$ $s = 18.4$ $N = 7$	—	$\bar{x} = 53$ ft.
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} \to \infty$	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				CEMENT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
				COLUMN MEANS	$\bar{x} \to \infty$	$\bar{x} \to \infty$		*Grand Mean* $\bar{x} \to \infty$
				IGNEOUS AGGREGATE	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
		10 IN.	NONE	WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$
300-400 PEN.	LOW VISCOSITY	4 INCHES	16 INCHES	WITHOUT FILLER	$\bar{x} = 110$ $s = 92.4$ $N = 4$	$\bar{x} = 37$ $s = 10.4$ $N = 8$	—	
				CEMENT FILLER	$\bar{x} \to \infty$	$\bar{x} = 43$ $s = 25.8$ $N = 7$	—	
				COLUMN MEANS				
SC-5				WITHOUT FILLER	—	$\bar{x} \to \infty$	—	$\bar{x} \to \infty$

$\bar{x}$ = Mean Crack Spacing *s = Standard Deviation*
N = Sample Size (No. Cracks − 1)

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TABLE 23

SUMMARY OF ALL TYPE CRACK SPACING

TRAFFIC LANE

SAND SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	ROW MEANS
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}=5.4$ $s=3.9$ $N=70$	$\bar{x}=4.4$ $s=2.8$ $N=88$	$\bar{x}=5$ $s=5.0$ $N=67$	$\bar{x}=5$ ft.
				CEMENT FILLER	$\bar{x}=8$ $s=5.2$ $N=41$	$\bar{x}=8$ $s=4.9$ $N=44$	—	$\bar{x}=8$ ft.
				COLUMN MEANS	$\bar{x}=6.5$ ft.	$\bar{x}=6$ ft.	$\bar{x}=5$ ft.	Grand Mean $\bar{x}=6$ ft.
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}\rightarrow\infty$	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$
				CEMENT FILLER	—	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$
				COLUMN MEANS	$\bar{x}\rightarrow\infty$	$\bar{x}\rightarrow\infty$		Grand Mean $\bar{x}\rightarrow\infty$
				IGNEOUS AGGREGATE	—	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
300-400 PEN.	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}\rightarrow\infty$	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$
				CEMENT FILLER	$\bar{x}\rightarrow\infty$	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$
				COLUMN MEANS	$\bar{x}\rightarrow\infty$	$\bar{x}\rightarrow\infty$		Grand Mean $\bar{x}\rightarrow\infty$
SC-5				WITHOUT FILLER	—	$\bar{x}\rightarrow\infty$	—	$\bar{x}\rightarrow\infty$

$\bar{x}$ = Mean Crack Spacing s = Standard Deviation
N = Sample Size (No. Cracks − 1)

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TABLE 24

SUMMARY OF ALL TYPE CRACK SPACING

PASSING LANE

SAND SUBGRADE SECTIONS

ASPHALT GRADE	ASPHALT TYPE	PAVEMENT THICKNESS	GRAVEL BASE COURSE	PAVEMENT MIX TYPE	BELOW OPTIMUM ASPHALT CONTENT	OPTIMUM ASPHALT CONTENT	ABOVE OPTIMUM ASPHALT CONTENT	*ROW MEANS*
150-200 PENETRATION	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}=7.1$ $s=6.2$ $N=53$	$\bar{x}=3.6$ $s=2.6$ $N=102$	$\bar{x}=5$ $s=4.4$ $N=58$	$\bar{x}=5$ ft.
				CEMENT FILLER	$\bar{x}=10$ $s=7.3$ $N=34$	$\bar{x}=7$ $s=5.0$ $N=48$	—	$\bar{x}=6$ ft.
				COLUMN MEANS	$\bar{x}=8.5$ ft.	$\bar{x}=5.5$ ft	$\bar{x}=5$ ft.	Grand Mean $\bar{x}=6$ ft.
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
150-200 PENETRATION	HIGH VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}\to\infty$	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$
				CEMENT FILLER	—	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$
				COLUMN MEANS	$\bar{x}\to\infty$	$\bar{x}\to\infty$		Grand Mean $\bar{x}\to\infty$
				IGNEOUS AGGREGATE	—	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$
		10 IN.	NONE	WITHOUT FILLER	—	—	—	
300-400 PEN.	LOW VISCOSITY	4 INCHES	6 INCHES	WITHOUT FILLER	$\bar{x}\to\infty$	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$
				CEMENT FILLER	$\bar{x}\to\infty$	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$
				COLUMN MEANS	$\bar{x}\to\infty$	$\bar{x}\to\infty$		Grand Mean $\bar{x}\to\infty$
SC-5				WITHOUT FILLER	—	$\bar{x}\to\infty$	—	$\bar{x}\to\infty$

$\bar{x}$ = *Mean Crack Spacing* s = *Standard Deviation*
N = *Sample Size (No. Cracks – 1)*

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Table 25 - Significance Levels at which Differences Between Mean Crack Spacing were not Significant for Traffic and Passing Lanes

SECTION NUMBER	SIGNIFICANCE LEVEL	
	TYPE 1 CRACKS	ALL CRACK TYPES
LV 150-200 PEN., 4-16-CLAY SECTIONS		
54	10%	10%
63	10%	10%
55	10%	10%
53	10%	10%
LV 150-200 PEN., 10-0-CLAY SECTION		
64	10%	10%
LV 150-200 PEN., 4-6-SAND SECTIONS		
74	10%	5%
67	10%	1%
77	10%	10%
76	10%	10%
75	10%	10%
LV 300-400 PEN., 4-16-CLAY SECTIONS		
58	1%	*
61	10%	10%
60	10%	10%

* Significant difference between means exists at 1% level.

between the mean crack spacing of the traffic and passing lanes is not significant at the 10 percent level. Thus, it may be concluded that traffic loading is not significant in contributing to transverse pavement cracking.

Effect of Asphalt Content and Cement Filler Content on Transverse Cracking

An examination of the transverse crack spacing data in Tables 17 to 24 for the low viscosity 150-200 penetration grade asphalt sections with the same type of subgrade did not appear to indicate a distinct difference in the mean transverse crack spacing among the below optimum and optimum asphalt content sections or among the no-filler and filler sections. To determine whether a significant difference existed among the means or to determine if the variation among the means simply reflected the variance in a parent population, analyses of variance using the F-statistic were carried out as described by Freund and Williams*. This method served to indicate whether the 'between sample variation' was greater than the 'within sample variation' and verified whether all of the samples belonged to the same population or to populations with diverse means.

As prior studies had indicated no significant differences in transverse crack spacing between the traffic and

*Freund, J. E., and Williams, F. J., "Elementary Business Statistics: The Modern Approach," Prentice-Hall, Inc., 1964, pp. 384-389.

the passing lanes, the transverse crack spacing data from each lane was treated as a separate sample in the analyses of variance among the sections of the same asphalt type and grade and pavement structure. The comparisons were made at significance levels of ten, five and one percent using the data from all of the related sections initially. Certain samples exhibited anomalous characteristics and they were excluded from ensuing analyses. The levels at which transverse cracking among related sections was not significant are tabulated in Table 26.

Table 26. Level at which Transverse Crack Spacing was not Significant Among Sections of the Same Asphalt Type and Grade and Pavement Structure

Asphalt	Structure	Type of Cracking	Significance Level	Remarks
LV 150-200 PEN.	4-16-CLAY	Type 1	10%	
		All Types	5%	
LV 150-200 PEN.	4-6-SAND	Type 1	10%	Pass. lanes Sec.67 & 76 excluded
		All Types	*	
LV 300-400 PEN.	4-16-CLAY	Type 1	10%	Sec. 59 excluded
		All Types	10%	Sec. 59 excluded

*Significant difference among means at 1% level.

The analyses of variance of the transverse crack spacing among the related sections shown in Table 26 showed no significant difference in Type 1 crack spacing at the 10 percent significance level. When all the type of cracks were considered, a reduction in the significance level resulted, indicating greater differences in the nature of the crack spacing among the sections. This may be attributed to the erratic nature of the occurrence of Type 2, 3 and 4 cracks in contrast to the tendency for Type 1 cracks to occur at regularly spaced intervals. However, no special import may be attached to this as the reduction in the significance levels does not reflect any systematized cracking behavior patterns among the sections with different asphalt contents or among the sections with different cement filler contents.

On the basis of the above observations it is concluded that neither asphalt content nor cement filler content were significant in affecting the frequency of transverse pavement cracking within the limits of the content ranges studied.

Effects of Asphalt Type and Grade on Transverse Cracking

Inspection of the means of the section crack spacings in Tables 17 to 24 indicated that the greatest difference in transverse crack spacing was between sections with different asphalts. As asphalt content and cement filler content were not significant in affecting transverse cracking, sections with similar structures but of the same asphalt type and grade were treated as samples from the same population and the means of the sections were averaged to give the best estimate of the population mean. The standard error of each population mean was determined from the formula

$$\sigma_{\bar{x}} = \frac{\sigma}{\sqrt{n}}$$

where:

$\sigma_{\bar{x}}$ = standard error of the population mean,

σ = standard deviation of the sample,

n = sample size.

The LV 300-400 PEN., 4-16-CLAY sections proved to be a special problem; the predetermined section length of 400 feet was insufficient to permit the observation of the required number of cracks in each section necessary to draw a reliable conclusion regarding the effect on transverse cracking performance by this asphalt. The four adjoining sections were pooled together and treated as a single sample to yield an estimate of the population mean. However, the crack spacing was too erratic to permit an estimate of the standard error of the mean.

The data summarized in Table 27 indicates that the asphalt type and asphalt grade variables have an implicit effect on the transverse cracking frequency of asphalt pavements. Comparisons of the means of sections with similar

Table 27. Effect of Asphalt Type and Pavement Structure on Transverse Cracking

Asphalt	Structure	Type 1 Cracks: Mean Crack Spacing (Ft.)	Type 1 Cracks: Std. Error of Mean	All Crack Types: Mean Crack Spacing (Ft.)	All Crack Types: Std. Error of Mean
LV 150-200 PEN.	4-16-CLAY	20	2.1	20	2.4
	4-6-SAND	8	2.2	6	1.9
	10-0-CLAY	50	3.5	45	8.0
LV 300-400 PEN.	4-16-CLAY	65		65	
	4-6-SAND	*		*	
HV 150-200	4-16-SAND	*		*	
	4-6-SAND	*		*	
	10-0-CLAY	*		*	
HV SC-5	4-16-CLAY	*		*	
	4-6-SAND	*		*	

*Pavement sections with no transverse cracking to date.

structures show that pavements with low viscosity type asphalts of the 150-200 penetration grade exhibit a significantly greater frequency of transverse cracking than high viscosity asphalts of the same grade. Similarly, pavements with "softer" low viscosity type asphalts, such as the 300-400 penetration grade, do not appear to be as susceptible to transverse cracking as pavements with "harder" low viscosity type asphalts, such as the 150-200 penetration grade. Neither the high viscosity type 150-200 penetration grade asphalt pavements nor the high viscosity type SC-5 asphalt pavements have showed signs of transverse cracking to date.

Effects of Pavement Structure Design on Transverse Cracking

The data in Table 27 indicates that the frequency of transverse pavement cracking varied inversely to the thickness of the asphaltic concrete layer. In the LV 150-200 PEN. asphalt pavement sections the mean Type 1 crack spacing was 20 feet in the 4-16-CLAY pavements while the 10-0-CLAY pavement section exhibited Type 1 cracking at an average of every 50 feet. A similar comparison could not be made between the HV 150-200 PEN., 4-16-CLAY and 10-0-CLAY pavements because neither has exhibited any transverse cracking to date.

Effects of Subgrade Type on Transverse Cracking

The data in Table 27 indicates that the frequency of transverse pavement cracking of the asphaltic concrete surface is affected by the type of subgrade soil. In the LV 150-200 PEN. asphalt sections the mean Type 1 crack spacing of the 4-16-CLAY pavements was 20 feet but 8 feet for the 4-6-SAND pavements. However, similar pavement structures, in which LV 300-400 PEN. asphalts were incorporated, exhibited the converse transverse cracking relationship with the 4-16-CLAY pavements exhibiting a greater transverse cracking frequency than the 4-6-SAND pavements. This difference in transverse cracking performance implies that there is an inter-action between the asphaltic concrete surface and the subgrade and that the frequency of transverse cracking is dependent upon the low temperature properties of these materials.

When all of the crack types were considered, a considerably greater reduction in the mean crack spacing was observed in the LV 150-200 PEN. asphalt, 4-6-SAND sections than in the 4-16-CLAY sections with the same asphalt. This indicated that the interaction between certain types of asphaltic concretes which are susceptible to transverse cracking and a sand subgrade at low temperatures may result in a greater amount of Type 2, 3 and 4 cracking.