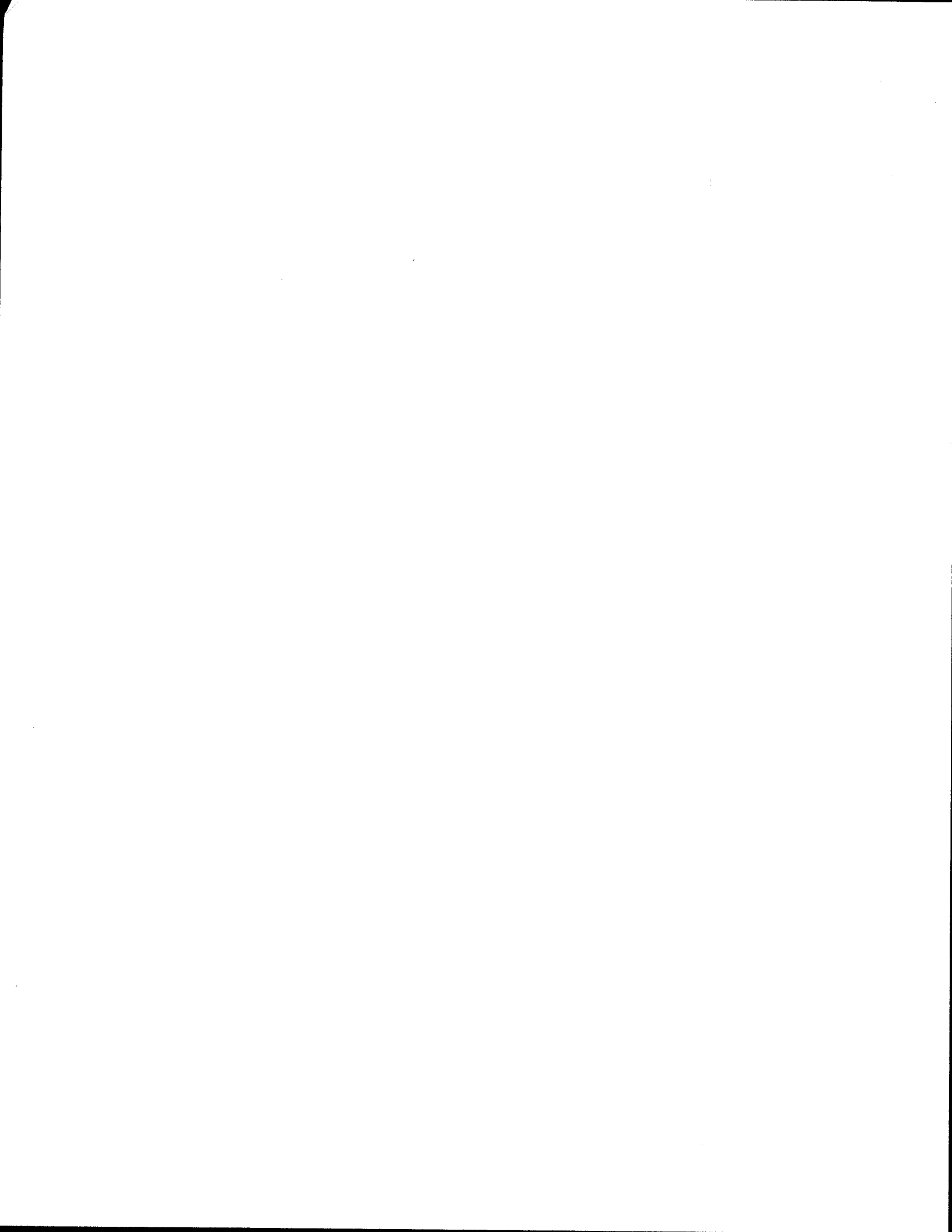


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16. Abstract <p>Intersection approach pavements often experience extreme forms of distress long before the tangent segments of the same pavement and long before the design life of the pavement is attained. Field and laboratory investigations of asphalt concrete intersection approach pavements were conducted to determine the primary causes of premature failure and suggest changes in pavement design and construction procedures that can be used to prolong intersection pavement service life.</p> <p>The primary mode of failure of the intersections studied was rutting with shoving in some cases. The leading materials related cause of pavement failure was asphalt content in excess of the designed value. Most of the mixtures studied contained relatively high percentages of uncrushed sand and low voids in the mineral aggregate.</p> <p>Modifications in materials specifications, laboratory test techniques design procedures, and construction methods are suggested to provide a margin of safety to minimize early failures. The potential for significant economic benefits appears promising when intersection approaches are designed and constructed to accommodate the special stresses to which they are subjected.</p>			
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AVOIDING EARLY FAILURE OF INTERSECTION PAVEMENTS

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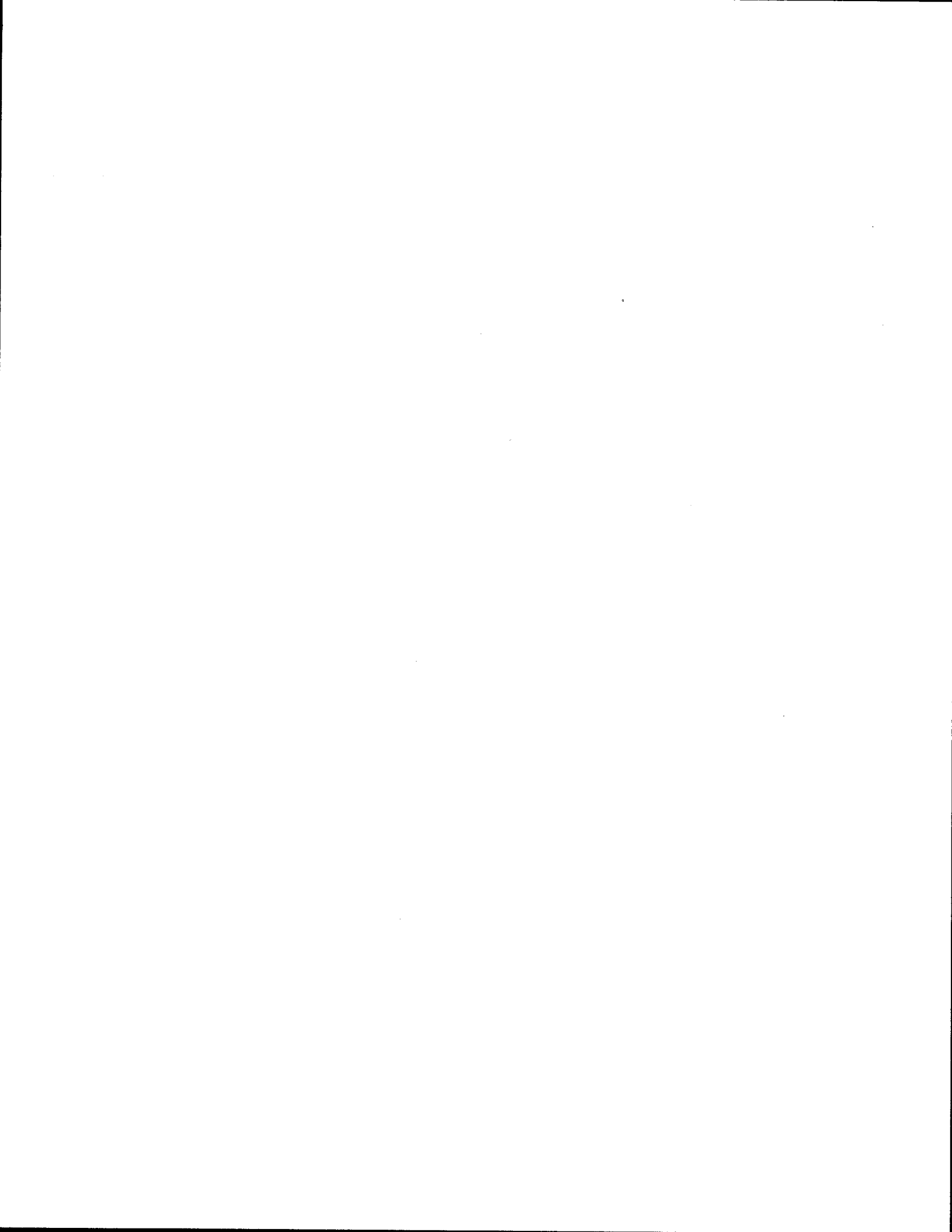
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College Station, Texas 77843

November 1989



METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km

AREA				
in ²	square inches	645.2	millimetres squared	mm ²
ft ²	square feet	0.0929	metres squared	m ²
yd ²	square yards	0.836	metres squared	m ²
mi ²	square miles	2.59	kilometres squared	km ²
ac	acres	0.395	hectares	ha

MASS (weight)				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg

VOLUME				
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft ³	cubic feet	0.0328	metres cubed	m ³
yd ³	cubic yards	0.0765	metres cubed	m ³

NOTE: Volumes greater than 1000 L shall be shown in m³.

TEMPERATURE (exact)				
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi

AREA				
mm ²	millimetres squared	0.0016	square inches	in ²
m ²	metres squared	10.764	square feet	ft ²
km ²	kilometres squared	0.39	square miles	mi ²
ha	hectares (10 000 m ²)	2.53	acres	ac

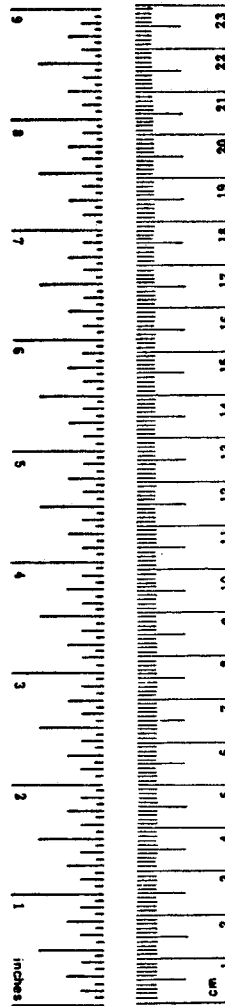
MASS (weight)				
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T

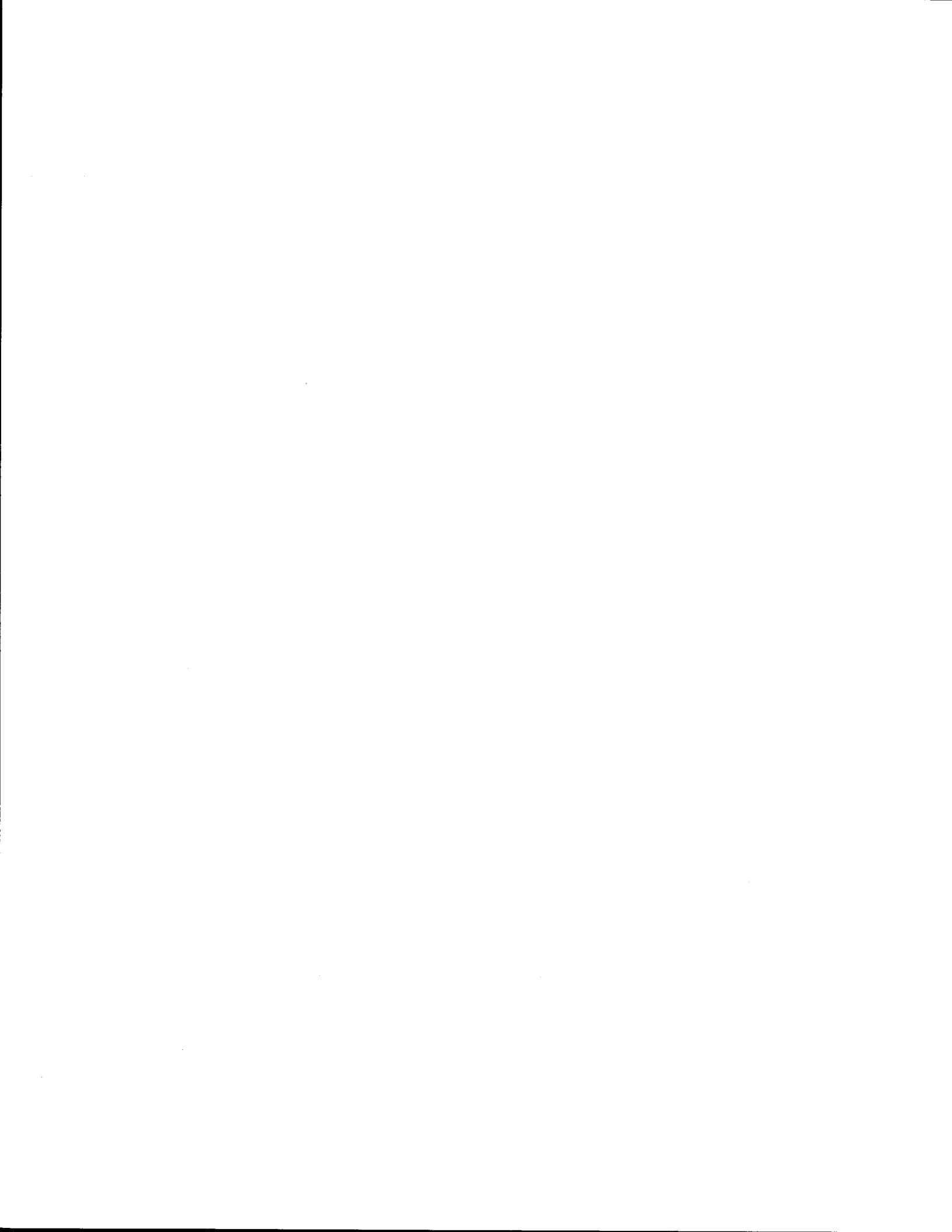
VOLUME				
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m ³	metres cubed	35.315	cubic feet	ft ³
m ³	metres cubed	1.308	cubic yards	yd ³

TEMPERATURE (exact)				
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements





IMPLEMENTATION STATEMENT

The findings of this study indicate that existing technology can be used to design and construct pavements of adequate strength and stability to withstand the special stresses associated with intersection approaches. This report recommends changes in existing materials specifications, laboratory test procedures, and asphalt mixture design methods in order to decrease the probability of premature failure of intersection pavements. It suggests that alternatives other than standard dense graded asphalt mixtures should be considered for construction of intersection approach pavements because these standard mixtures are neither designed to withstand the special stresses applied at intersections nor have they proved to be successful in intersection applications. Initial costs of implementing improved intersection designs will be significantly more than those encountered in normal practice. However, use of the improved designs and/or paving materials may show a significant cost savings during the designed service-life resulting from reduced spot maintenance and user costs associated with maintenance activities. This will be particularly true for high volume roadways.

The most common mode of asphalt pavement distress encountered at intersections was rutting with some shoving and flushing. Primary materials related sources of these problems were asphalt in excess of the designed value, high percentages of glassy, uncrushed natural sands, and/or very dense-graded mixtures with low voids in the mineral aggregate (VMA). Dense mixes with low VMA and high sand contents are generally quite sensitive to small changes in binder content. Variation in binder content due to mix plant operations or upward adjustments by Department personnel in order to achieve specified density will cause these mixes to exhibit instability under loading by traffic.

Suggested modifications to Item 340 specifications designed to increase mixture toughness include such items as: reduction of sand-size particles, further limitations on uncrushed aggregate, addition of a VMA requirement, and increasing minimum Hveem stability. The use of large stone mixtures with a maximum aggregate size of 3 inches (or up to 2/3 of the pavement layer thickness) containing asphalt additives to increase binder viscosity at high pavement service temperatures is recommended. A specification for "washed" crusher

screenings (manufactured fines) should be established as these materials will be required to replace the natural, uncrushed or rounded sand particles that need to be eliminated.

Other recommendations include the use of (1) the 0.45 power aggregate gradation chart to improve control during mixture design and construction, (2) a rational approach for intersection mixture design to increase the probability of success, (3) constant asphalt viscosities during laboratory mixing and compacting rather than constant temperatures to reduce the probability of specifying excess binder when hard asphalts are used, and (4) a sequential construction technique where all intersection approaches within a project are built or overlaid using a special paving mix prior to placement of the tangent or connecting sections.

An alternative approach to eliminate plastic flow of the pavement surface materials at intersections is the application of portland cement concrete.

Optimum length of the special intersection pavement will depend on level of traffic, traffic control methods and, of course, the average length of the queue line that forms during stoppages. Evidence indicates the typical length will range from 100 to 250 feet.

Results of this work may be implemented to provide adequate structures in other specially stressed segments of pavements such as bus terminals, steep vertical and horizontal curves, and even airport runways and taxiways.

DISCLAIMER

The contents of this report reflect the view of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specifications, or regulation.

There is no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

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INTRODUCTION

BACKGROUND

The AASHTO Guide for Design of Pavement Structures (1) states that "It is worth noting again that while the Guide describes and provides a specific method which can be used for the determination of alternate design or rehabilitation recommendations for the pavement structure, there are a number of considerations which are left to the user for final determination." Standard pavement structural design methods including those presented in the Guide and asphalt mixture design procedures were developed for pavements with moving traffic without regard for high, repetitive shear stresses, such as those generated by decelerating and accelerating heavy vehicles at certain pavement locations. Traffic loading, often expressed as passages of an 18 kip equivalent single axle load (ESAL), as determined from the AASHTO Road Test, are used in the calculation of damage factors to estimate design life of a pavement. By definition, the ESAL's are applied by freely rolling tires which principally apply a vertical load to the pavement with the only horizontal load in the pavement being the force component generated by the vertical load.

Unique forces are experienced by pavements in certain nontangent areas. For example, vehicles approaching an intersection apply the brakes to decelerate, often at very rapid rates, thus applying tremendous longitudinal forces in the direction of travel at the pavement surface. They come to a stop and apply vertical loads to the pavement for extended periods of time. Then they accelerate and the drive wheels apply significant longitudinal forces in the direction opposite to travel. These stresses impart a dynamic kneading action to bituminous pavements and thus induce permanent deformation (rutting) at faster rates than in tangent sections of identical pavements. In addition, the horizontal shear forces applied to the pavement surface by braking vehicles in concert with the rolling action shove the asphalt mixture longitudinally. This action results in the formation of corrugations which are defined as transverse undulations at regular intervals in the surface of a pavement consisting of alternate valleys and crests. Corrugations are sometimes referred to as "wash board" pavement. Furthermore, these facilities receive comparatively large deposits of

Lubricating oil and fuel from motor vehicles which may soften the binder.

The AASHTO Guide (1) also states, "The designer will need to concentrate on some aspects of design which are not always covered in detail in the Guide." There is a need to analyze the horizontal shear forces unique to certain portions of pavement systems and develop design procedures, specifications and materials acceptance criteria which can be used to prolong pavement service life and reduce maintenance/rehabilitation activities in these specially stressed portions of pavement.

STATEMENT OF PROBLEM

Asphalt concrete pavements are typically designed and built as if the complete paving project was a tangent section. For this reason, nontangent segments of a pavement very often experience extreme forms of distress long before the tangent segments of the pavement and long before the design life of the pavement is attained. As a result, maintenance and/or rehabilitation of the specially stressed segments is required early in the pavement's service life which is costly both from the materials and labor standpoint as well as the user cost standpoint.

More specifically, specially stressed portions of pavements such as intersections, curves, approaches to railroad crossings, bus terminals and steep grades are exposed to horizontal forces that are many times greater than those on tangent sections and to vertical forces that are often applied for much longer periods. Distress that appears early in the pavement's service life usually manifests itself in asphalt concrete pavements as longitudinal and lateral movement and/or consolidation of the paving mixture and in seal coats as flushing.

These types of pavement distress will often result in hazardous driving conditions. For example, in approaches to intersections, corrugations and ruts can develop in asphalt concrete to the extent that vehicle control is adversely affected. Consolidation of the paving mixture can produce flushing, resulting in a slick surface and/or driver complaints due to asphalt on their automobiles. Intersections inherently provide much more potential for danger than tangent segments of a given roadway. Furthermore, these hazardous conditions are compounded during periods of darkness and/or wet weather.

Nontangent segments of pavement, particularly on high traffic volume roads, should be designed and built to withstand damaging stresses in order to provide service lives approximately equivalent to those of adjacent tangent sections. This can be accomplished using current technology. Cost-effectiveness of special treatment of these pavement sections during construction should be investigated on an individual basis.

PURPOSE AND SCOPE

This study addresses the initial phases of the problem as described above. The analysis is limited to asphalt concrete surfaced intersections. The overall purpose of this study is to develop techniques that can be employed in a cost-effective manner to design and build specially stressed portions of pavements that will exhibit performance equivalent to the tangent sections. Specific objectives include the following:

1. Estimate the magnitude of the problem of premature intersection failure.
2. Estimate the horizontal and vertical stress distributions generated by decelerating and accelerating vehicles.
3. Recommend mixture designs, using state-of-the-art technology, that are capable of withstanding the applied stresses with acceptable levels of damage.
4. Suggest pavement materials acceptance criteria, evaluation methods, construction techniques and inspection policies that will maximize intersection pavement quality.



FIELD INVESTIGATION

SELECTION OF INTERSECTIONS

A questionnaire (Appendix A) with a brief description of the study was distributed among all the Texas highway districts in order to locate unsuccessful asphalt concrete intersections. Unsuccessful intersections were defined as those exhibiting significant pavement distress-related problems, such as rutting and/or corrugations, within two years after construction. Replies listing unsuccessful intersections were received from District 10 (Tyler), District 18 (Dallas), District 19 (Atlanta), and District 20 (Beaumont). Visual inspections were performed on about 20 unsuccessful intersections and eight were selected for further study. Some of these turned out to be more than 2 years old.

Those districts indicating they had unsuccessful intersections, which were considered by the researchers to be candidates for this study, were asked if they also had successful intersections. Successful intersections were defined as those exposed to reasonably heavy traffic and exhibiting less than 0.25 inches of rutting and insignificant corrugations and/or flushing after 4 or more years of service. When few successful intersections were found in these districts, the request for successful intersections was sent to other districts. Successful intersections were reported in eight additional districts. Visual inspections were made on approximately 30 of these of which 7 were considered for further study. Most of the intersections that were reported to be successful actually exhibited significant distress or they experienced very low traffic levels which eliminated them from study. A sufficient number of "good" intersection approaches which had received their last overlay more than 4 years ago were not found. Therefore, some of the good intersections selected for study had received an overlay less than four years prior to this evaluation. Successful candidate intersections were found in Districts 8 (Abilene), 13 (Yoakum), 15 (San Antonio), 18 (Dallas) and 19 (Atlanta).

It was found that many districts have implemented an intersection maintenance program in which basically all intersections in the district exposed to significant traffic are upgraded every other year, as a minimum.

Although the program is doing an excellent job in maintaining intersection quality, it did cause some difficulty in locating candidate intersections for this research study.

DESCRIPTION OF INTERSECTIONS SELECTED FOR STUDY

After performing a visual inspection on all the proposed sites, specific intersections were selected for further analysis. Figure 1 shows the location of the selected intersections. Some intersections were sampled and tested while others were given a more cursory study. Where possible, mixture design data, materials descriptions, typical sections and a sampling of daily construction reports were obtained. Rutting was found to be the primary mode of distress in all unsuccessful intersections identified. A summary of the intersections selected for study is given in Table 1.

SAMPLING AND TESTING PROGRAM

Rut depths were measured on the approach side of the intersections from the cross street and back until ruts were less than 0.125-inches deep. Twenty-five 4-inch diameter cores were obtained from the rutted areas of selected intersections. At the approach side of the intersections, five cores distributed across the pavement, in and between the wheelpaths, were drilled in order to ascertain the profile of the transverse cross section of the pavement. Cores were drilled in accordance with this scheme at each of 5 different locations to obtain a total of 25 cores. The cores were conveyed to the laboratory where the surface layer portions were carefully separated by sawing and later tested (Figure 2) in an attempt to identify the possible causes of pavement distress.

In some instances, the cores were found to consist of a series of up to 8 thin (less than 1 inch) layers of asphalt concrete pavement. Mixture testing of these cores was not performed. Testing of these in situ materials and interpretation of resulting data would have been very difficult. In these cases, testing was limited to visual inspection and the

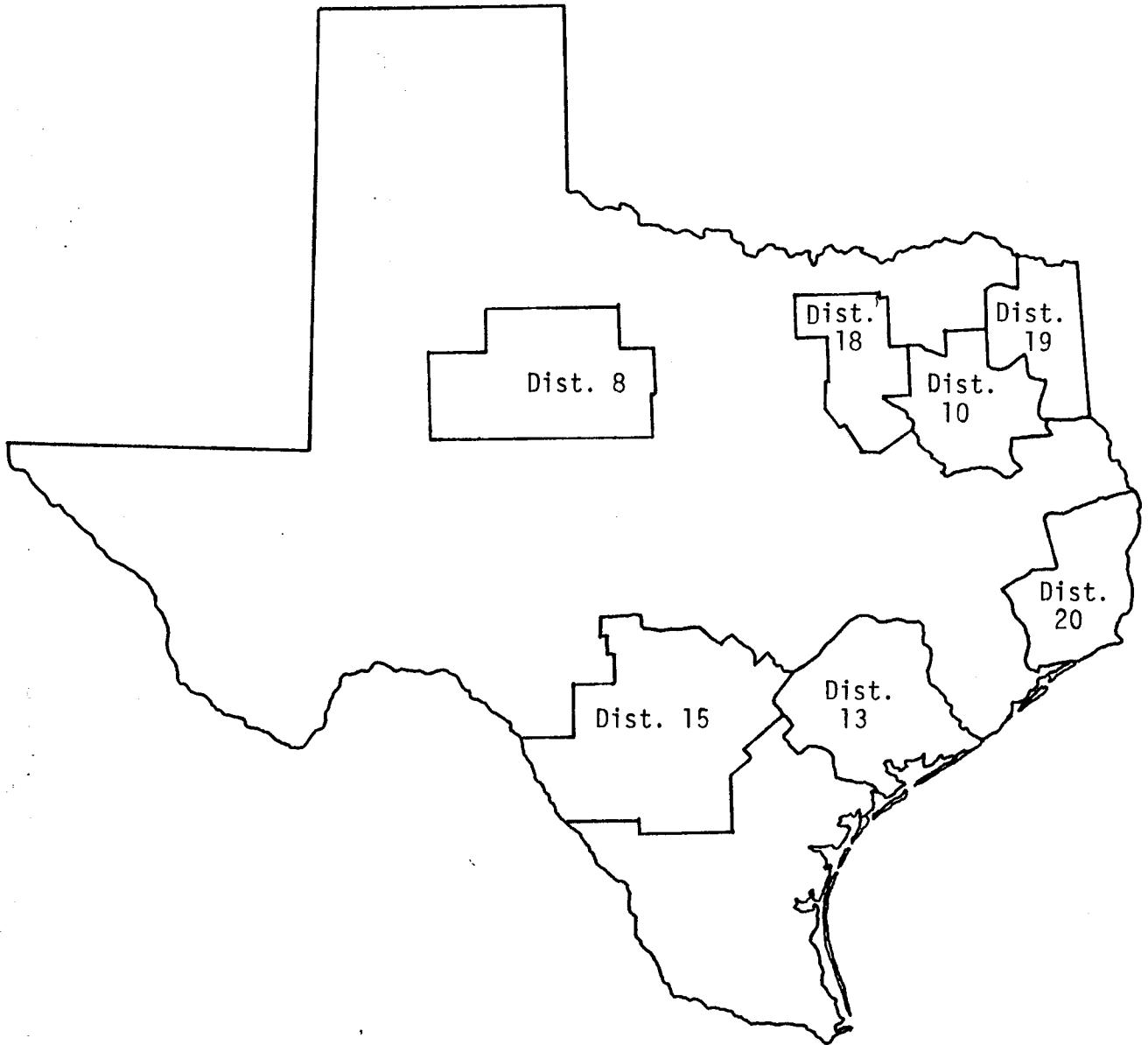


Figure 1. Location of Selected Intersections.

Table 1. Summary of Selected Intersections

Location	Identification	Traffic, ADT,	Age of Last Overlay	Rut Depth, in.	Other Distress
District 8 Abilene	SH 36 @ Judge Ely*	4,600	6 1/2 yr	<0.25	None
District 10 Tyler	Loop 323 @ FM 756	38,000	5 mo	0.75-0.9	Flushing
	Loop 323 @ Mackim		5 mo	0.5-0.9	Flushing
	Loop 323 @ SH110		5 mo	<0.25	None
District 13 Yoakum	SH60 @ SH36*	2,000	3 yr		None
District 15 San Antonio	Toepperwein @ IH35*	12,000	2 yr		None
	Judson @ IH35*	12,000	5 yr	<0.10	Slight Flushing
	Collesieum @ IH35*	10,000	5 yr	0.05	None
District 18 Dallas	FM2170 @ SH5	18,800	4 yr	0.25-1.0	Shoving
	SH66 @ Rowlett*	14,000	3 yr	<0.2	None
District 19 Atlanta	US259 @ SH11	8,000	8 yr	0.13-1.0	None
	US67 @ FM989	6,700	9 yr	0.3-0.9	Shoving
	US59 @ FM989*	19,000	8 yr	0	None
District 20 Beaumont	US96 @ FM1013	10,000	6 yr	0.25-2.5	Shoving
	US190 @ US96	10,100	2 yr	0.13-1.0	Shoving

* Indicates good intersections

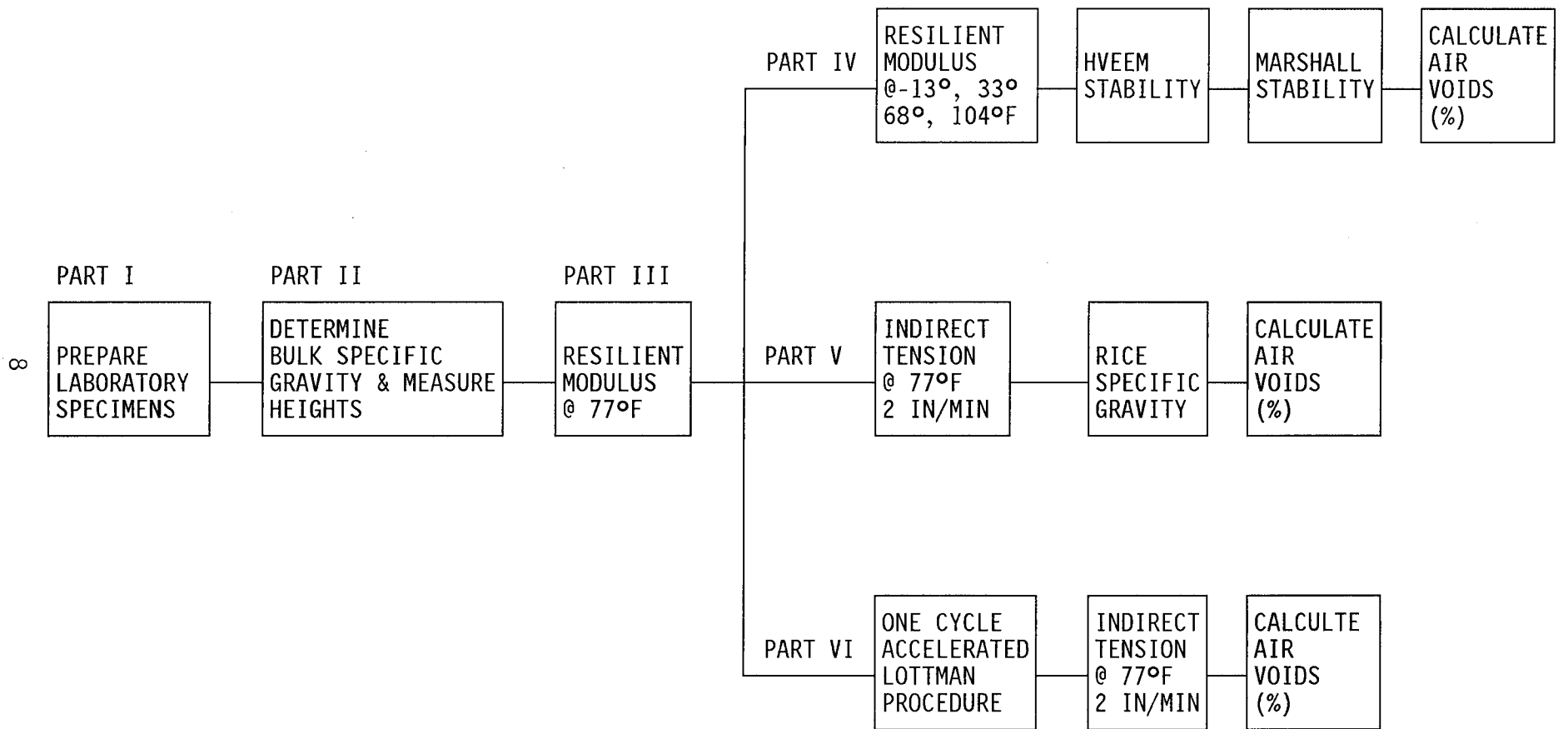


Figure 2. Laboratory Test Program.

following tests on the top two layers:

1. Layer depths, an attempt to ascertain which layer(s) was responsible for the permanent deformation,
2. Air void content,
3. Asphalt content,
4. Asphalt viscosity,
5. Aggregate gradation, and/or
6. Aggregate classification.

TEST RESULTS

Laboratory test results are described in the following subsections for each of the intersections analyzed. The test results are separated by district. Rut depths, gradations, air voids, voids in the mineral aggregate, aggregate characteristics and asphalt contents are analyzed and compared.

District 10

Test results for the intersections in District 10 are reported in Tables 2 through 4 and Figure 3. Tables 2 and 3 summarize the laboratory mixture test results. Table 4 shows a comparison between the mixture design data and measurements following extraction of asphalt from the cores. Figure 3 shows the aggregate gradations measured using pavement cores and the maximum density or Fuller Curve.

Rut depths were measured only 5 months after mixture was placed.

Mixture Properties. The air voids measured (2-5 percent) were relatively low for a pavement of this age. Voids in the mineral aggregate (VMA) appeared acceptable since they are within the criteria specified by the Asphalt Institute (2), which recommends a minimum of 16 percent VMA for a mixture containing 3/8-inch maximum particle size (Figure 4) (3). However, low air void content with VMA that is within specified limits is an indicator of excess asphalt. This excess asphalt will decrease the

Table 2. Properties of Cores (Uppermost Layer) From Intersections in District 10.

Test Plan Part*	Rutted Site	Wheel Path	Air Void Content, percent	VMA, percent	Resilient Modulus, psi x 10 ^{6**}				Hveem Stability	Marshall Stability, lbs.	Marshall Flow, 0.01inch
					77°F	68°F	33°	-13°F			
IV	Yes	Yes	4.0	17.2	0.116	0.228	1.006	1.502	-	1462	13.7
V	Yes	Yes	3.7	16.9	0.123	-	-	-	-	-	-
IV	Yes	No	2.3	15.7	0.184	0.327	1.113	1.550	23.1	2096	9.5
V	Yes	No	2.5	15.9	0.191	-	-	-	-	-	-
VI	Yes	No	3.7	16.9	0.171	-	-	-	-	-	-
IV	No	Yes	2.3	14.9	0.297	0.447	1.040	1.876	-	2509	9.5
V	No	Yes	2.4	15.1	0.379	-	-	-	-	-	-
IV	No	No	3.5	16.0	0.362	0.470	1.303	1.738	54.5	1540	9.5
V	No	No	3.4	15.8	0.318	-	-	-	-	-	-
VI	No	No	4.7	17.0	0.322	-	-	-	-	-	-

10

* See Figure 2

** Specimens were too tender to measure resilient modulus at 104°F

Notes: Each result for rutted intersections is average of three values

Each result for nonrutted intersection is average of two values

Table 3. Tensile Properties of Cores (Uppermost Layer) From Intersections in District 10.

Test Plan Part*	Rutted Site	In Wheel Path	Indirect Tension Before Lottman				Indirect Tension After Lottman				
			Air Void, percent	Tensile Strength, psi	Strain @ Failure, percent	Secant Modulus, psi	Air Void, percent	Tensile Strength, psi	Strain @ Failure, percent	Secant Modulus, psi	Tensile Strength ratio
V & VI	Yes	No	2.5	223	0.585	46,000	3.7	137	0.469	30,000	59
V & VI	No	No	3.4	226	0.313	72,000	4.7	145	0.309	47,000	64

* See Figure 2

Note: - Each result for rutted intersections is average of three values

- Each result for nonrutted intersections is average of two values

Table 4. Design vs. Extracted Data for Intersections Overlay in District 10.

DESIGN DATA:

Mix Type: D
 Specification: Item 340
 Materials Used: Crushed Sandstone 65%
 Sandstone Screenings 35%
 Asphalt AC-20 Exxon
 Asphalt Content: 6%
 VMA: 17% (From Actual Sp. Grav.)
 Air Voids: 5.2% (From Actual Sp. Grav.)
 % Minus # 200: 2.5%

EXTRACTED DATA:

	Intersection A * (Rutted)	Intersection B * (Rutted)	Intersection C * (Nonrutted)
Asphalt Content, percent	6.6	6.5	6.0
VMA, percent	15 - 18	16 - 18	15 - 17
Air Voids, percent	2 - 4	3 - 5	2 - 5
Percent Minus # 200	4.3	4.3	4.3
Asphalt Viscosity at 140°F, poise	2000	30	70
Asphalt Penetration	69	50	36
Pen/Vis Number	-0.62	-0.27	-0.78

* Intersection A: Loop 323 & FM 756
 Intersection B: Loop 323 & Mackim
 Intersection C: Loop 323 & Texas 110

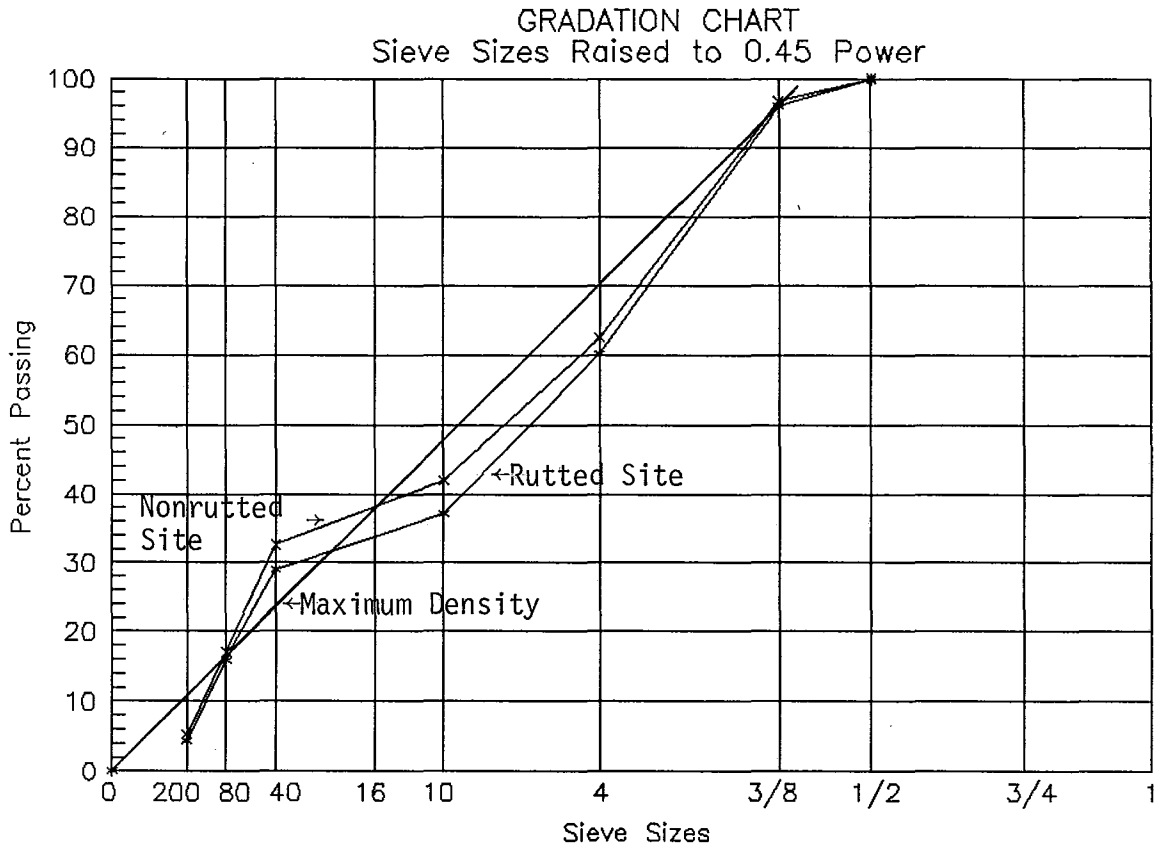


Figure 3. Gradations of Aggregates from Cores Obtained from Rutted and Unrutted Sites on Loop 323 in Tyler - District 10.

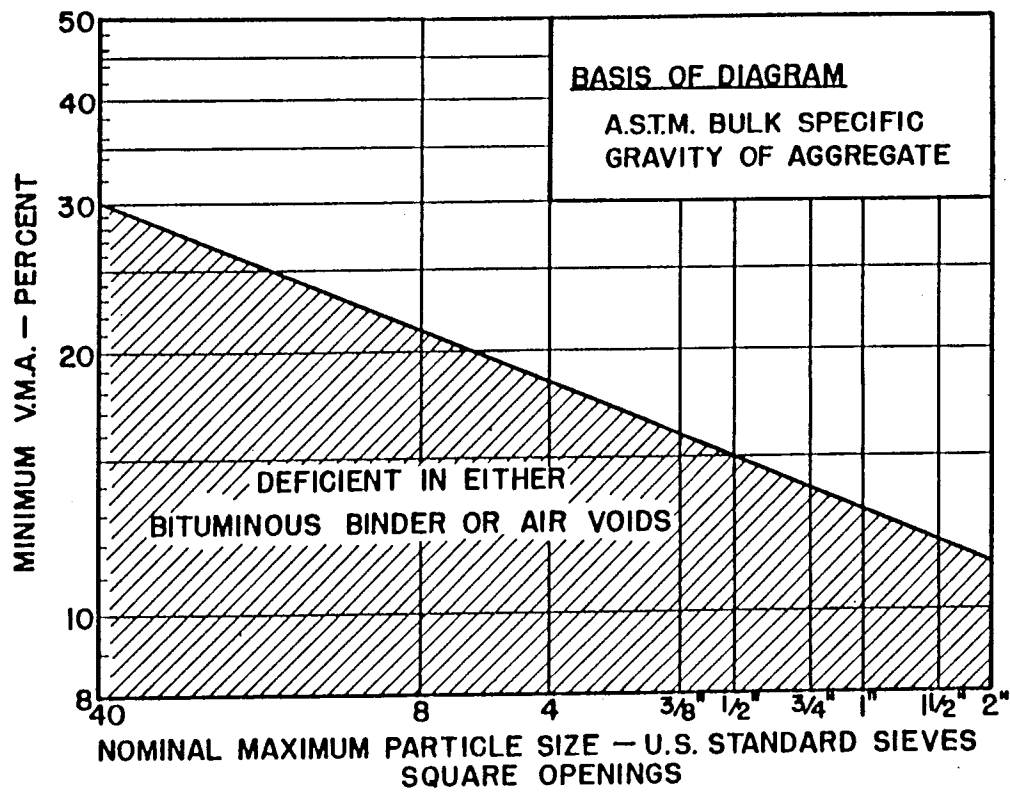


Figure 4. Minimum VMA vs Maximum Particle Size (After McLeod (3)).

internal friction of the mixture, making it unstable under slow moving or stationary traffic loads, particularly during hot weather on a newly placed pavement.

All the cores were extremely tender at higher test temperatures and, as a result, collapsed when attempts were made to perform the resilient modulus test at 104°F. Stiffness of the mixtures from the rutted sites were consistently lower than those from the nonrutted sites as evidenced by resilient modulus at 77°F and lower. In addition, Hveem stability was much lower for the cores obtained from the rutted sites. Marshall stability and flow, on the average, also exhibited more critical values from the rutted sections as compared to the nonrutted section. These differences can only be explained by the higher asphalt content of the mixtures from the rutted sites and the fact that the mixture appears to be very sensitive to binder content.

Aggregate Properties. The plotted gradations (Figure 3) show a slightly more dense mix for the rutted intersection approaches as compared to the nonrutted intersection approach. This is based on comparison with the maximum density or Fuller Curve. However, these differences may not be significant as only one sample was tested to obtain each curve. In addition, a notably high percentage of sand-size particles is indicated by the hump at the No. 40 sieve in the gradation curve. The aggregate system is composed of 100 percent crushed sandstone. The coarse aggregate was angular and rough in texture. However, upon examination under the microscope, the fine aggregate was found to consist of a high percentage of individual sand particles which appeared to be mostly subangular, glassy and nonporous. Apparently, the sandstone is not well cemented and upon crushing, a significant portion reverted to the original individual sand particles.

Asphalt Properties. Asphalts were extracted from the cores and penetration and viscosity were measured at 77°F and 140°F, respectively. The results (Table 4) indicate that the asphalt used at Intersection A (Loop 323 and FM 756) was an AC-10 instead of an AC-20. Measurements of asphalt

content showed that the mixtures from the rutted intersections contained about 0.5 percent more asphalt than optimum.

Analysis. The major contributor to failure of this intersection mixture was the excess asphalt content. Excess asphalt created the low void content. The glassy, nonabsorptive character of the aggregate and the relatively low filler (minus #200 aggregate) content made the mixture sensitive to asphalt content and, therefore, increased the propensity for permanent deformation problems. This sensitivity to binder content may have been relieved significantly by the incorporation of limestone crusher fines.

District 20

Rut depths at the intersection approach of US 96 at FM 1013 in Kirbyville measured 0.75 to 2.5-inches. Nearest the intersection, where the vehicles halted, a ridge had developed alongside the outer edge of the outside wheelpath. Rut depths at the approach of US 190 at US 96 in Jasper measured 0.13 to 1.0-inches. The pavements more than 250 feet back from the intersections appeared to be in good condition with rut depths less than 0.125-inches.

After examining the cores, it was concluded, by matching layer profiles with the rut depths measured, that the pavement consisted of a succession of overlays, each of which had experienced various degrees of rutting. The layers within the cores were approximately 1 inch thick and thus too thin to accommodate most of the standard mixture tests.

Only the uppermost overlay was tested. Test results for District 20 are reported in Tables 5 and 6 and Figures 5 and 6. Table 6 shows a comparison between the mixture design data and measurements made following extraction of asphalt from the cores. Figures 5 and 6 show the aggregate gradation measured from pavement cores and compared to the maximum density or Fuller Curves.

Table 5. Properties of Uppermost Layer of Cores From Intersections in District 20.

Site	Wheel Path	Air Voids, percent	VMA percent
Kirbyville (US 96 at FM 1013)	Yes	2.1	14.3
	No	4.8	16.7
Jasper (US 190 at US 96)	Yes	1.9	15.4
	No	4.7	17.8

Note: Each value is average from three tests.

Table 6. Design vs. Extracted Data for Mix Applied on Intersections in District 20.

<u>FIELD DESIGN DATA:</u>	<u>Kirbyville</u>	<u>Jasper</u>
Date of Construction:	April, 1982	September, 1986
Mix:	C	D
Specification:	340-086	340-075
Materials Used:	Sandstone 28%	#78 Coarse Limestone 23%
	Limestone 19%	#8 Limestone 37%
	Screening 17%	Limestone Screening 10%
	Field Sand 36%	Field Sand 30%
	Asphalt: Texaco AC-20	Asphalt: Texaco AC-20
Asphalt Content:	5.0%	5.3%
Air Voids:	3.6%	3.0%
Minus # 200:	2.2%	1.9%
 <u>EXTRACTED DATA:</u>		
Asphalt Content:	5.8%	5.4%
VMA:	14.3%(WP)* - (NWP)*	15.4%(WP) - 17.8%(NWP)
Air Voids:	2.1%(WP) - 4.8 (NWP)	1.9%(WP) - 4.7%(NWP)
% Minus # 200:	8.0%	8.5

*WP - Wheel Path

NWP - Not in Wheel Path

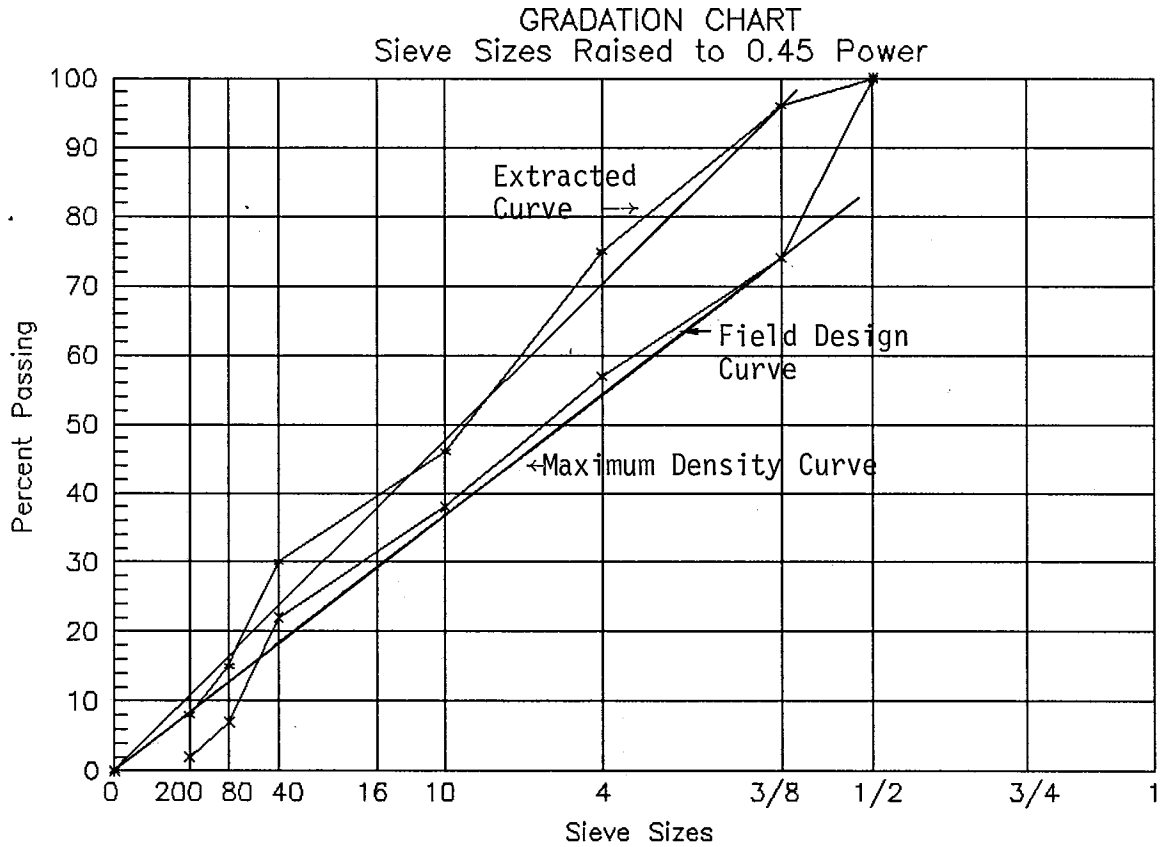


Figure 5. Mix Design and Extracted Gradations for Surface Mixture on Intersection Approach on US 96 at FM 1013 in Kirbyville - District 20.

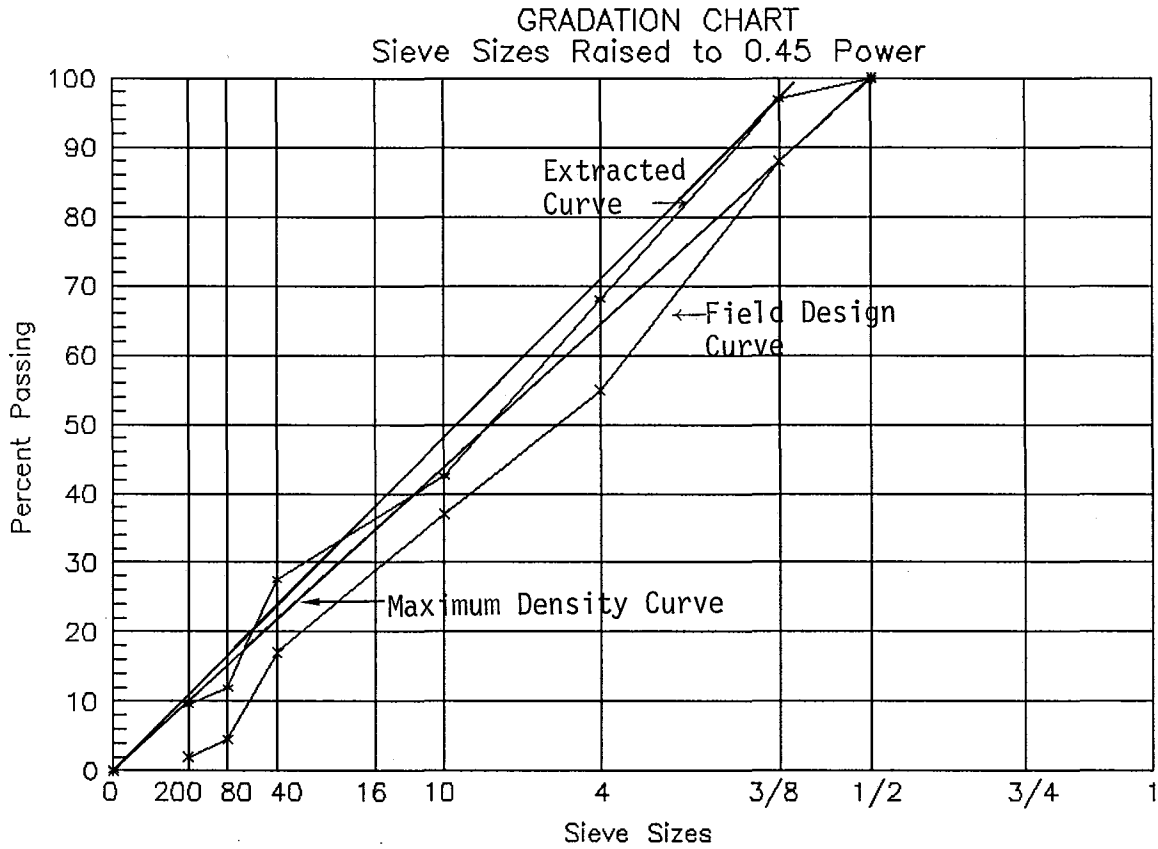


Figure 6. Mix Design and Extracted Gradations for Surface Mixture on Intersection Approach on US 96 at US 190 in Jasper - District 20.

Mixture Properties. Table 5 shows that considerable densification has occurred in the wheel paths at both intersections. Densification occurred rapidly in the pavement at Jasper, since it has been in service for only two years. The voids in the mineral aggregate are within the range specified by the Asphalt Institute (Figure 4).

Aggregate Properties. The aggregates blended to produce these mixtures contained 30 percent or more field sand (natural, uncrushed sand) by design. Based on results of sieve analyses (Figure 5 and 6), both mixtures were generally composed of aggregate significantly smaller in size than that specified by the design. In fact, the aggregate grading of the mixture from Kirbyville is closer to a Type D than a Type C. It is recognized that the coring operation will reduce the measured aggregate size, but not to the extent shown here, especially in the smaller size range. In addition, the gradation curve exhibited a notable hump at the No. 40 sieve, indicating an excess of sand and thus a mixture relatively weak in shear strength and sensitive to a slight excess of asphalt. Upon examination under the microscope, the fine aggregate was found to be mostly subangular to subrounded, showing smooth to polished surfaces and a nonporous siliceous character. The gradations measured (Figures 5 and 6) do not correspond well to the design gradations.

Asphalt Content. Extraction tests showed that the mixture from Kirbyville contained 0.8 percent more asphalt than the design; whereas, the mixture from Jasper contained very near the design content.

Analysis. A combination of high field sand content and overall small aggregate size produced a mixture susceptible to plastic flow. This problem was compounded at Kirbyville with the excessive asphalt. In time, traffic further densified the in-place mixtures to a low void level which further decreased its shear strength in the wheelpath and failure occurred due to rutting. Lateral flow of the surface mixture was evident in Kirbyville by the ridges alongside the wheelpaths near the intersection. It is believed that replacing part of the field sand with crushed particles and, of course,

careful control of asphalt content would provide a mixture much more resistant to plastic flow.

District 18

Test results from two intersections in District 18 are reported in Tables 7 and 8 and Figures 7 and 8. Table 7 shows some of the mixture properties. Table 8 shows the aggregate classification. Figures 7 and 8 show the aggregate gradation measured from pavement cores and the maximum density or Fuller Curves.

Measured rut depths in the intersection of FM 2170 at SH 5 were 1 inch or less. SH 66 at Rowlett street showed no signs of significant distress. Mix design data for these pavements was not available at the writing of this report. Both pavements were composed of a series of thin (less than 1 inch) overlays placed over a period of several years. Therefore, only a few tests were performed on the pavement cores.

Air voids in the uppermost pavement layer were very low (1 to 2.5 percent) for the rutted intersection and quite acceptable (5-7 percent) for the unrutted intersection.

The gradation of the surface layer in the unrutted intersection was coarser (Type C) than that of the rutted intersection (Type D). Reference Figures 7 and 8. The presence of the larger stones in the surface layer may have been a significant factor in its resistance to plastic deformation, as the composition of the subsequent layers and other factors such as traffic and subgrade were quite similar. In the minus number 40 sieve sizes, the aggregates in the surface layers of both pavements were largely subrounded, smooth textured and nonporous (Table 8).

District 19

Test results for materials from District 19 are reported in Tables 9 and 10 and Figures 9 through 11. Table 9 shows mixture properties from all three intersections. Table 10 shows the aggregate classifications. Figures 9 through 11 show the aggregate gradations measured from pavement cores. All mixtures tested were Type D.

Table 7. Properties of Uppermost Layer of Cores From Intersections in District 18.

Site	Rutted Site	Wheel Path	Air Voids, percent	Extracted Asphalt Content, percent
FM 2170 at SH 5	Yes	Yes	1.7	4.9
		No	2.6	4.9
SH 66 at Rowlett	No	Yes	5.7	4.5
		No	6.7	4.5

Note: Each result is an average from three values.

Table 8. Aggregate Classification of Uppermost Layer - District 18.

Site	Sieve Sizes	Shape	Texture	Porosity
FM 2170 at SH 5	+ # 40	Angular	Rough	Porous
	- # 40	Subrounded to Subangular	Smooth to Polished	Nonporous
SH 66 at Rowlett	+ # 40	Angular	Rough	Porous
	- # 40	Subrounded to Subangular	Smooth to Polished	Nonporous

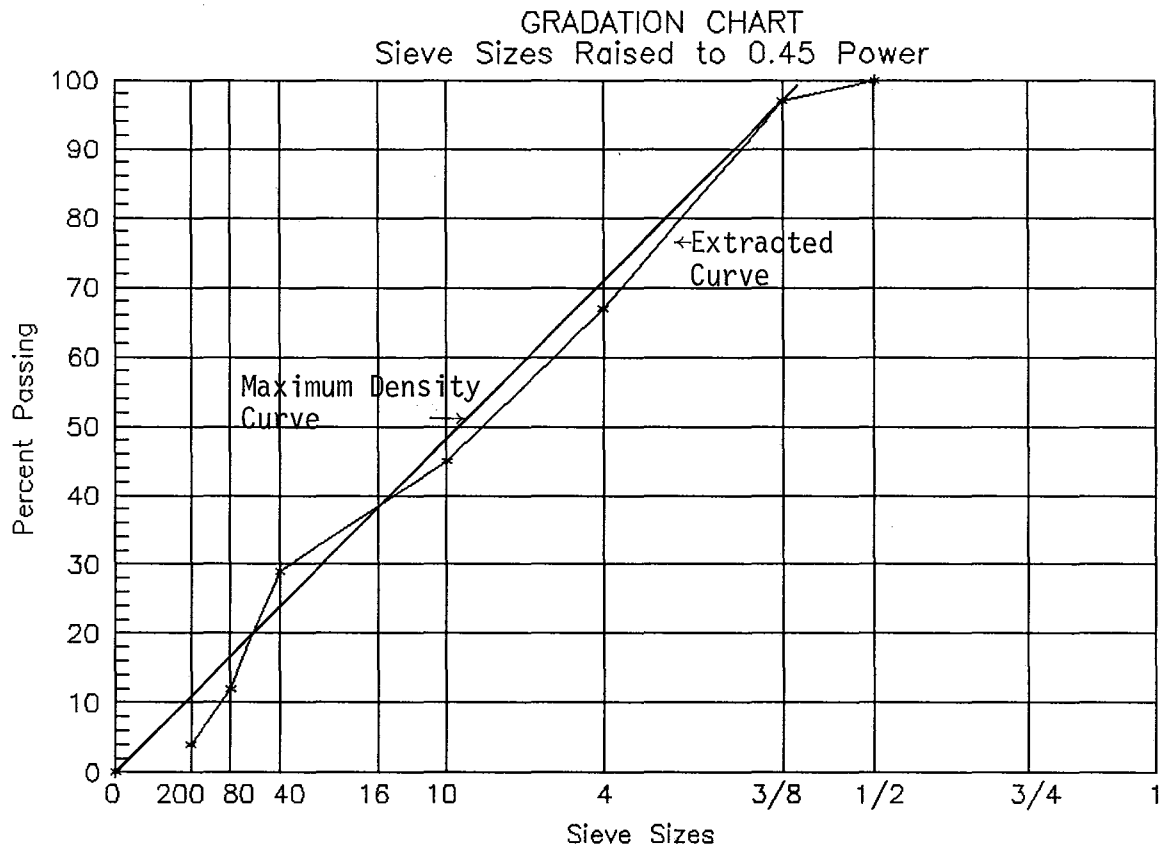


Figure 7. Gradation of Extracted Core for Surface Mix on Intersection Approach on FM 2170 at SH 5 in District 18.

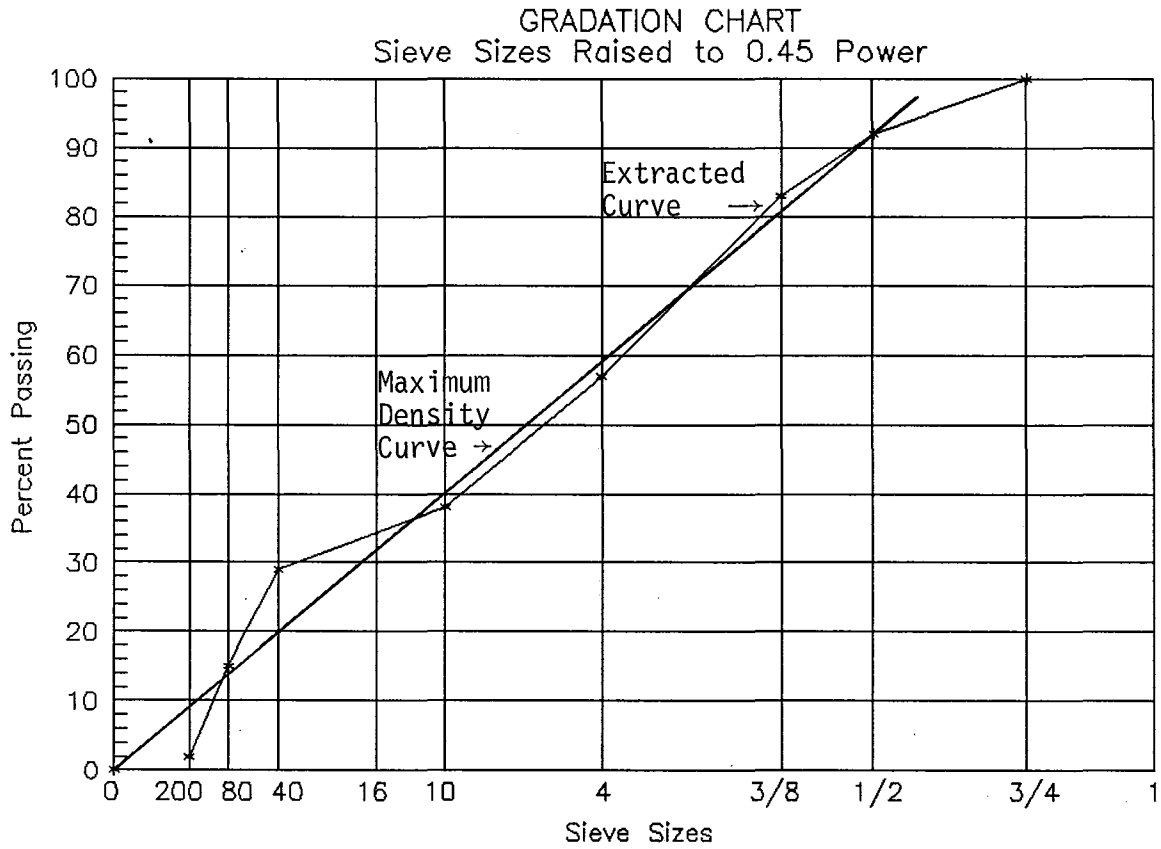


Figure 8. Gradation of Extracted Core for Surface Mix on Intersection Approach on SH 66 at Rowlett Street in Dallas - District 18.

Table 9. Properties of Upper Layers in Cores From Intersections in District 19.

	Location				
	US 259 @ SH 11		US 67 @ FM 989		US 59 @ FM 989
Rutting	<u>Yes: 1.0-in.</u>		<u>Yes: 0.9-in.</u>		<u>No: 0-in.</u>
Layer	<u>1</u>	<u>2</u>	<u>1</u>	<u>2</u>	<u>1</u>
Air Voids in Wheelpath, in.	1.5	1.4	1.8	1.4	5.4
Air Voids outside Wheelpath, in.	2.9	-	2.8	3.2	6.5
Asphalt Content, percent	5.7	5.9	6.9	4.7	5.1
Asphalt Viscosity at 140°F, poise	2950	-	-	-	-
Penetration at 77°F, dmm	65	-	-	-	-
Tensile Strength, psi	241	-	-	-	-
Tensile Strength, after Lottman, psi	177	-	-	-	-
Tensile Strength Ratio	73	-	-	-	-
Design Asphalt Content, percent	5.8	-	5.7	5.2	6.0
Hveem Stability for Mix Design	47	-	41	36	45
Aggregate Blend, percent					
Crushed Stone	58	-	65	-	60
Pea Gravel	-	-	-	50	-
Sand	25	-	25	35	20
Crusher Screenings	17	-	10	15	20

Table 10. Aggregate Classification - District 19.

Site	Rutted Site	Layer Analyzed	Sieve Sizes	Shape	Texture	Porosity
Daingerfield (US 259 at SH 11)	Yes	Top (# 1)	+ 40 - 40	Angular Subrounded	Rough Smooth	Porous Nonporous
		# 2	+ 10	Subrounded	Smooth	Nonporous
			10 - 40 - 40	Angular Subrounded	Rough Smooth	Porous Nonporous
Texarkana (US 67 at FM 989)	Yes	Top (# 1)	+ 40 - 40	Angular Subrounded	Rough Smooth	Porous Nonporous
		# 2	+ 40 - 40	Subrounded Subrounded	Smooth Smooth	Nonporous Nonporous
Texarkana (US 59 at FM 989)	No	Top	+ 40 - 40	Angular Subrounded	Rough Smooth	Porous Nonporous

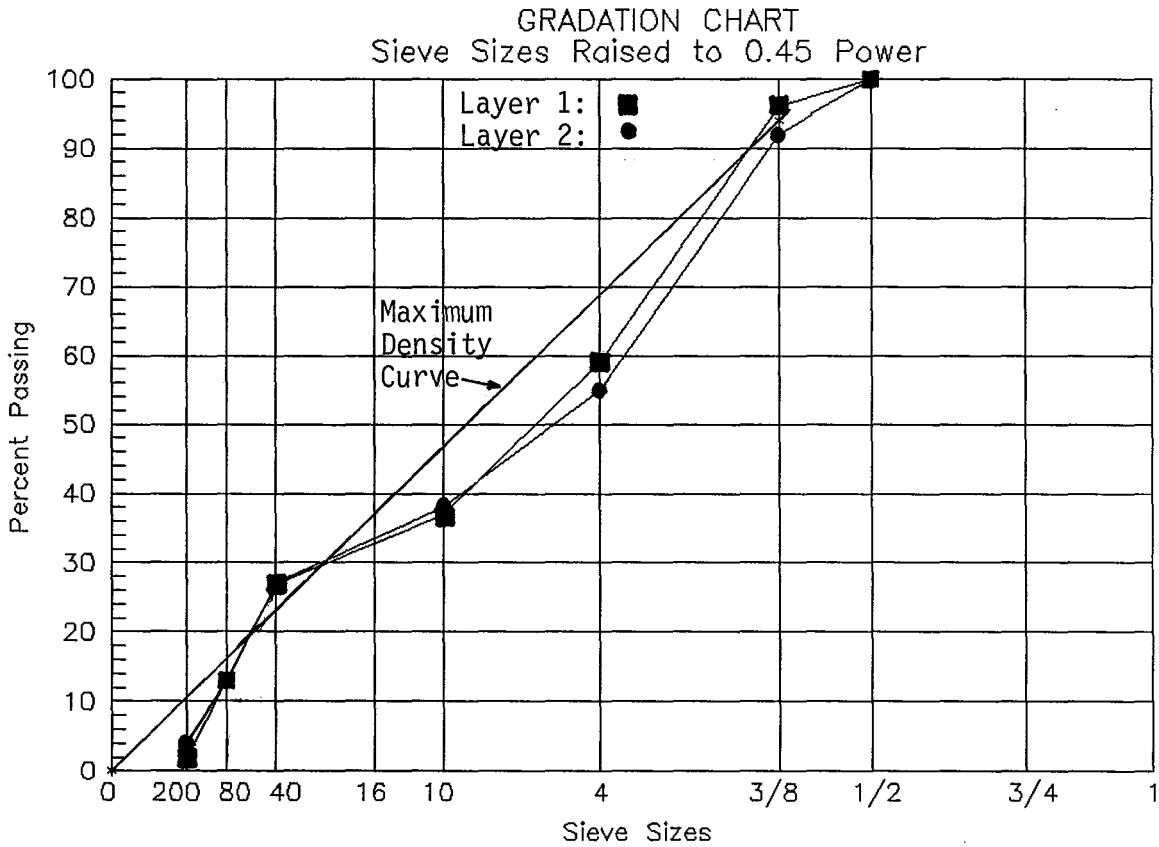


Figure 9. Gradation of Extracted Cores for Surface Mix and Second Layer Intersection Approach on US 259 at SH 11 near Daingerfield - District 19.

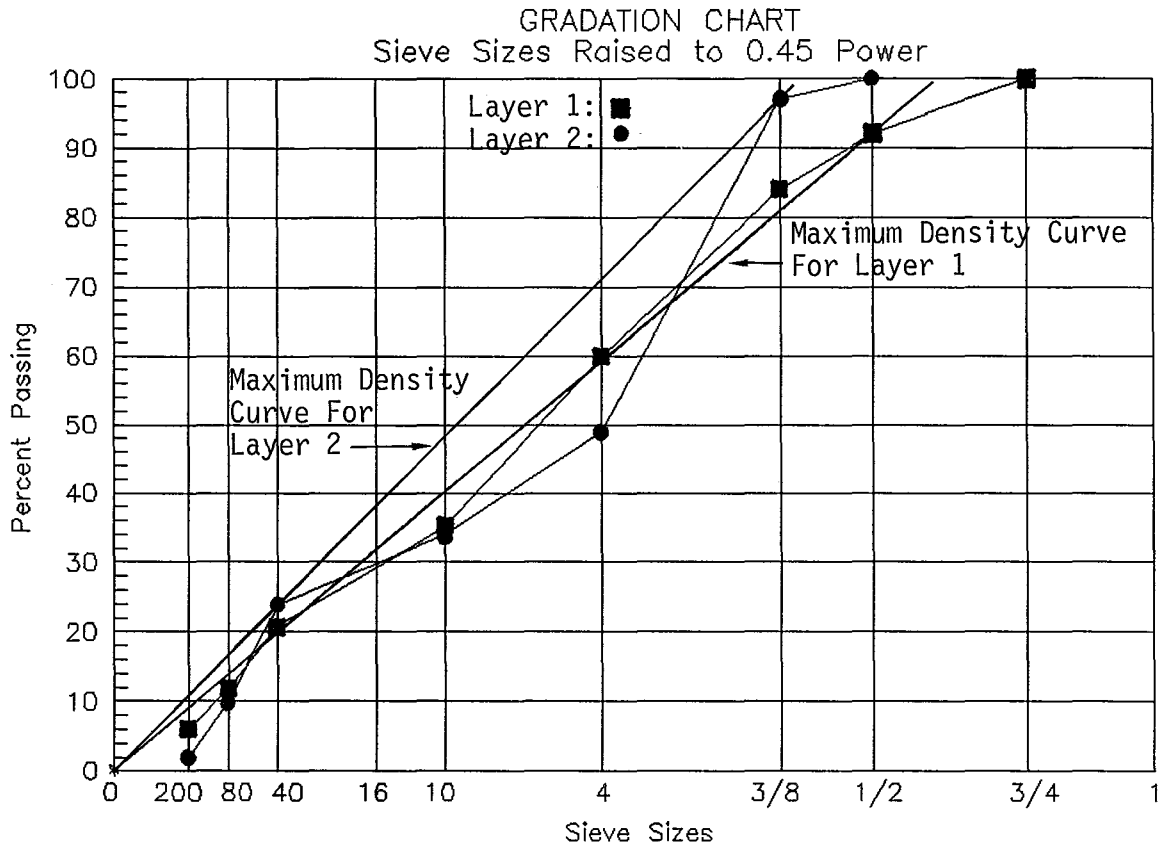


Figure 10. Gradation of Extracted Cores for Surface Mix and Second Layer on Intersection Approach on US 67 at FM 989 near Texarkana - District 19.

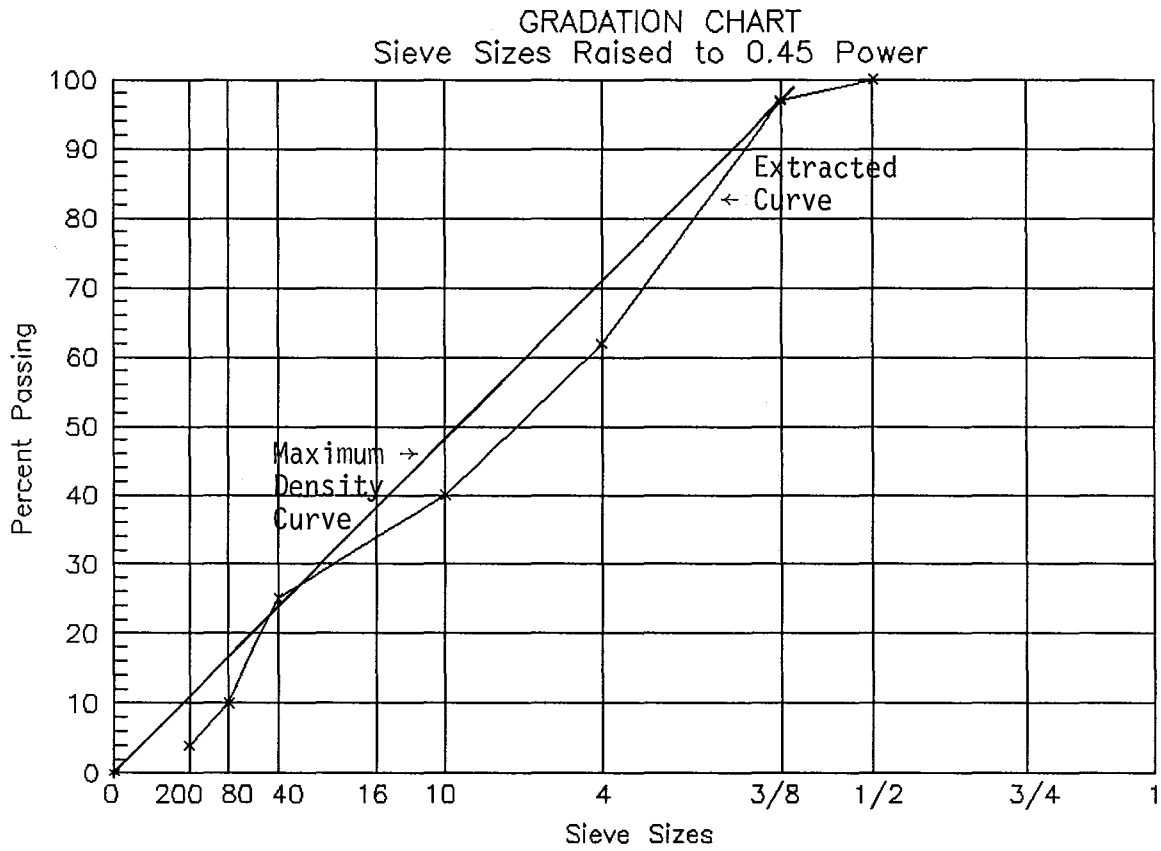


Figure 11. Gradation of Extracted Core for Surface Mix on Intersection Approach on US 59 at FM 989 near Texarkana - District 19.

Measured rut depths were between 0.13 inches and 1 inch for the intersection of US 259 at SH 11, and between 0.25 and 0.9 inch for the intersection of US 67 at FM 989. Researchers originally understood that these pavements were about two years old; it was later determined that they were considerably older. By that time, a significant amount data had been generated; therefore, they were included in the study. The intersection of US 59 at FM 989 exhibited no rutting or other forms of distress.

The upper two layers of the pavement cores from the rutted intersections were separated and tested to determine air void content, asphalt content and aggregate characteristics. The top two layers of the two rutted intersection approaches exhibited significantly lower air void contents than the upper layer of the unrutted intersection approach (Table 9). Air voids in the rutted pavements were extremely low in the wheelpaths (about 1.5 percent) as compared to the wheelpath of the unrutted pavement (5.4 percent) and quite low outside the wheelpaths.

Measured asphalt contents in the uppermost layers were somewhat higher for the rutted sections than for the unrutted section. In the rutted portion of US 67 at FM 989, the measured binder content in the uppermost layer exceeded the design value by 1.2 percent. On the contrary, in the unrutted pavement on US 59 at FM 989, the measured binder content was 0.9 percent less than the design value.

The aggregates in the upper layer of all three sections were crushed sand stone and field sand. Material in the plus number 40 sizes from the top layers of all three intersections was angular and rough textured (Table 10). However, the minus number forty material in all mixtures tested (both layers where applicable) was subrounded, smooth and nonporous. The plus number 40 material in the second layer of the rutted intersections was mostly pea gravel and was also subrounded, smooth and nonporous. The pea gravel layer of FM 67 at FM 989 was measurably thicker in the cores taken outside the wheelpath than in those from the wheelpath which indicates that plastic flow (rutting) had occurred and may have yet been occurring in this layer.

Although there was no visually observable evidence of stripping, a water treatment test using the Lottman (4) procedure revealed these highly silicious materials were fairly sensitive to damage by moisture (Table 9).

District 15

Three excellent intersection pavements on relatively high traffic volume facilities were found in San Antonio (District 15) (Table 1). Two of these pavements had been in service for five years and one had been in service for two years and neither of them showed any visible signs of distress. Each of the pavements was placed as new construction in two 1-inch lifts. Mixture design data for these two mixes is shown in Table 11.

These mixtures were composed of 74 percent crushed stone of various types and 26 percent field sand (Figures 12 and 13). However, it should be pointed out that the field sand was of exceptionally good quality in that the particles were angular to subangular and well-graded. The quantity of minus no. 200 sieve size material was comparatively low at about 3 percent. Asphalt contents were also comparatively low at 5 percent or less. However, asphalt film thickness in the mixtures at Toepperwein and Judson calculated to be more than 9 microns which is adequate for protection against moisture and oxidation.

Three of the four mixtures yielded laboratory compacted specimens with 15 percent VMA, as specified by the Asphalt Institute. The initial field compacted air void content ranged between 6.4 and 8.0 percent which apparently proved to be adequate for this mix. This combination of factors yielded excellent performance at these busy intersections which carried more than 10,000 vehicles per day.

District 13

An intersection surface mixture that had performed comparatively well for three summers was reported by District 13 personnel (Table 1). It is located at Wallis on SH 60 at SH 36 and carries about 2,000 vehicles per day. This Type D HMAC overlay was placed in a single lift of approximately 1½-inches in thickness. Mixture design data is provided in Table 12.

Table 11. Mix Design Data for Good Intersection Pavements in District 15.

Design Data	Location			
	Toepperwein/Judson at IH 35		Colliseum Rd at IH 35	
Layer Identification	1 (Surface)	2	1 (Surface)	2
Layer Thickness				
Specification Item	340	340	340	340
Mix Type	D	D	D	D
Aggregate Blend, percent				
Crushed Limestone	36	33	36	33
Crusher Screenings	7	7	7	7
Crushed Gravel	-	-	29	33
Crushed Sandstone	29	33	-	-
Field Sand	28	27	28	27
Absorption, percent	<1	<1	<1	<1
Minus # 200, percent	3.0	3.0	3.8	3.1
L. A. Abrasion, percent	30	30	30	30
Asphalt Source	Exxon	Exxon	Exxon	Exxon
Asphalt Grade	AC-20	AC-20	AC-20	AC-20
Asphalt Content	5.0	4.5	4.3	4.5
Avg. Specimen Density (Field), percent	96.5	96.5	97.1	96.7
Initial Avg. Field Voids, percent	6.4 - 8.0	6.4 - 8.0	-	-
Average Hveem Stability	46	46	40's	40's
VMA, percent	15.0	15.0	13	15.0
Minus # 200, percent	3.0	3.0	3.8	3.1

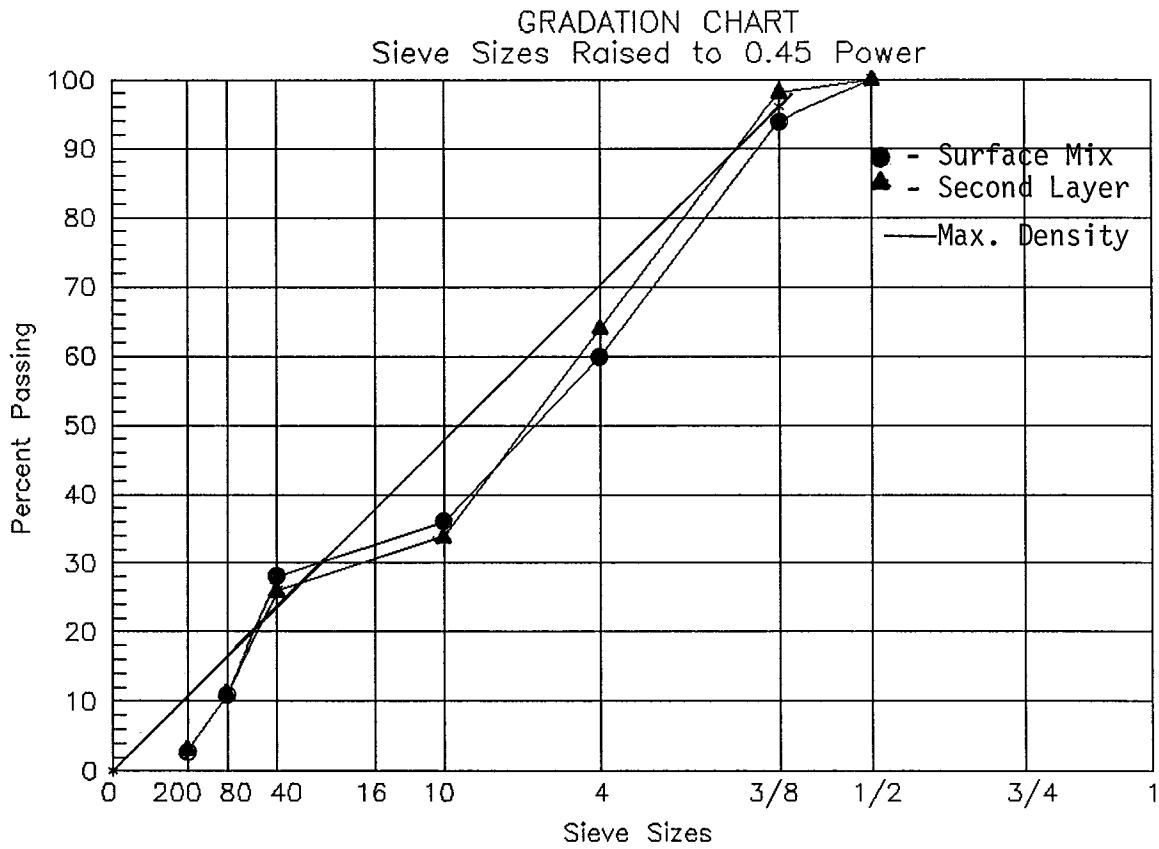


Figure 12. Design Gradation for Surface Mixture and Second Layer on Intersection Approaches at Toepperwein and Judson at IH 35-District 15.

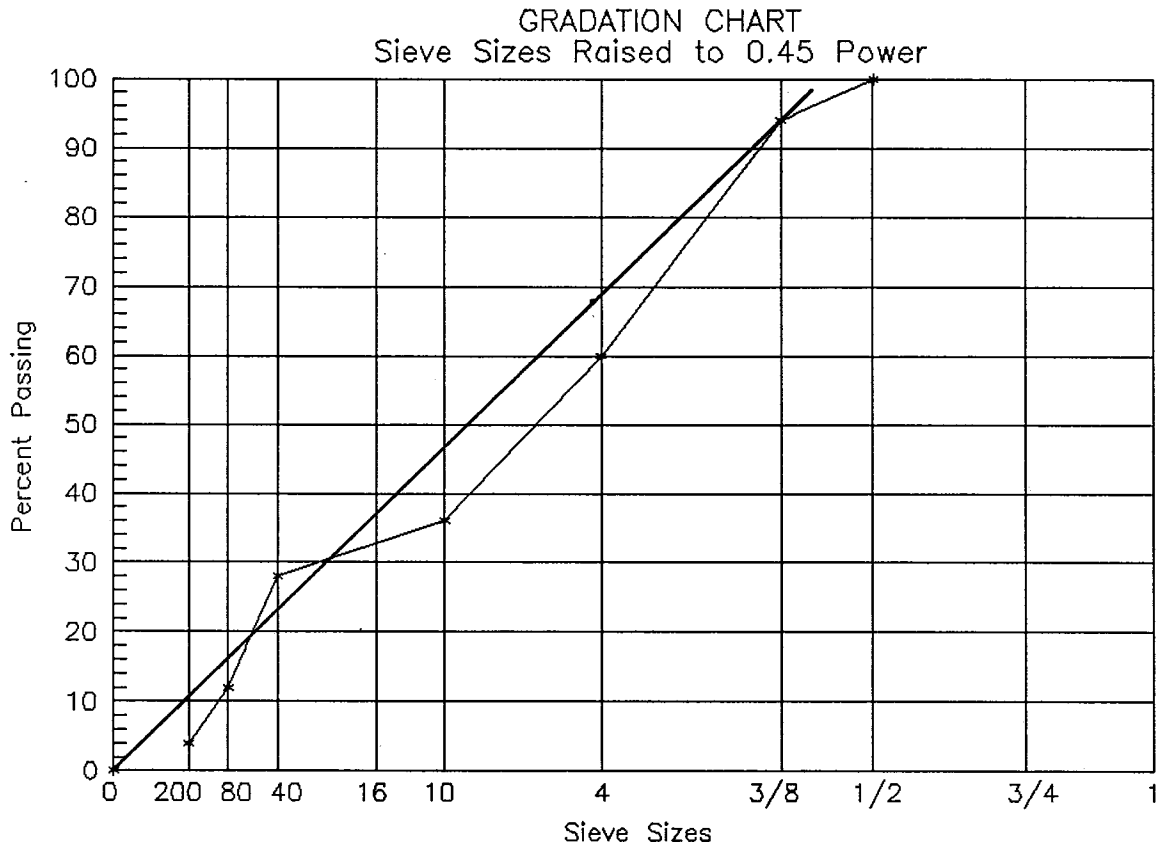


Figure 13. Design Gradation for Surface Mixture on Intersection Approach for Coliseum Road @ IH 35 - District 15.

Table 12. Mix Design Data for Good Intersection Pavement on SH60 at SH36 in Wallis - District 13.

District No.	13	8
Location	SH 60 at SH 36	SH 36 at Judge Ely
Item - Type	340 - D	340 - D
Aggregate Blend		
Crushed Limestone, percent	63	62
Limestone Screenings, percent	19	26
Field Sand	18	12
Minus # 200, percent	3.0	4.0
L. A. Abrasion (Coarse), percent	30	30
Asphalt	Texaco AC-20	Cosden AC-10
Asphalt Content, percent	4.7	6.2
Avg. Specimen Density (Field), percent	96.9	97.4
Hveem Stability	58	51
VMA, percent	12.1	-

Asphalt content was designed at 4.7 percent and, according to a sampling of daily construction reports, little variation was experienced indicating good quality control. The aggregate was comprised of 82 percent crushed limestone and 18 percent field sand with a total of 3.0 percent passing the no. 200 sieve (Figure 14). Although the VMA (12.1 percent) was lower than normally recommended by other agencies, the Hveem stability was quite high at 58 percent indicating very good interlock of rough textured aggregate and no excess of binder. This mix may be sensitive to binder content but, apparently, the design content was just right and quality control was sufficient to maintain the asphalt content to a satisfactory level to avoid problems with permanent deformation.

District 8

An intersection exhibiting excellent performance after six years in service in Abilene was reported by District 8 personnel. It is located on SH 36 at Judge Ely street and is exposed to an ADT of about 4,600 (Table 1). The surface mix was a Type D HMAC overlay placed in a single 1½-inch lift (Table 12). Visual inspection revealed no signs of plastic deformation, flushing, or any other forms of distress.

The aggregate was composed of 88 percent crushed limestone and 12 percent field sand. The filler (minus # 200) content was 4.0 percent. This rather fine-grained gradation is shown in Figure 15. The design asphalt content was 6.2 percent which yielded an average Hveem stability of about 51 percent. Field air voids after initial compaction were about 6 percent. The angularity of the coarse aggregate and the low field sand content are partially credited with the satisfactory performance of this intersection pavement. Six percent voids at initial compaction provided sufficient VMA to accommodate the rather high asphalt content and yet yielded adequate stability to prevent permanent deformation.

SUMMARY OF FINDINGS

Based on the findings from this field investigation, the following

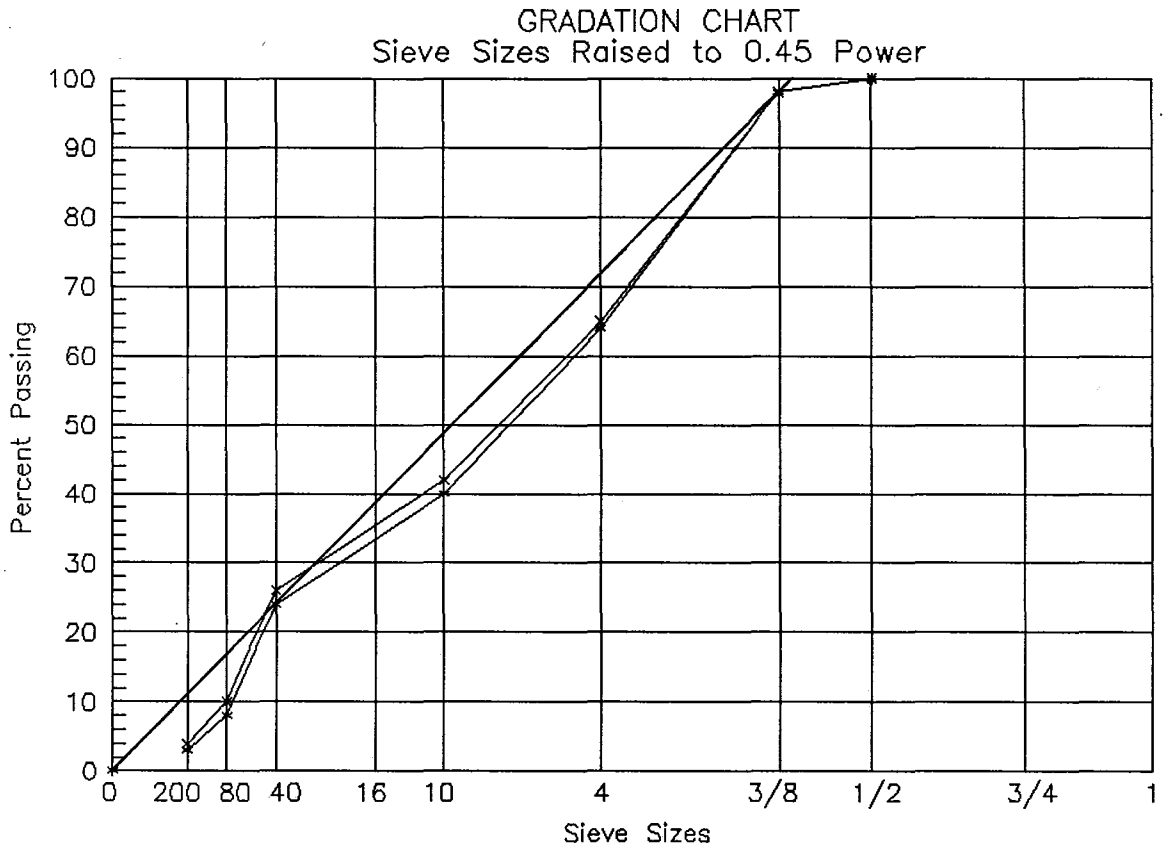


Figure 14. Design and Extracted Gradations for Surface Mixture for Intersection Approach on SH 60 at SH 36 - District 13.

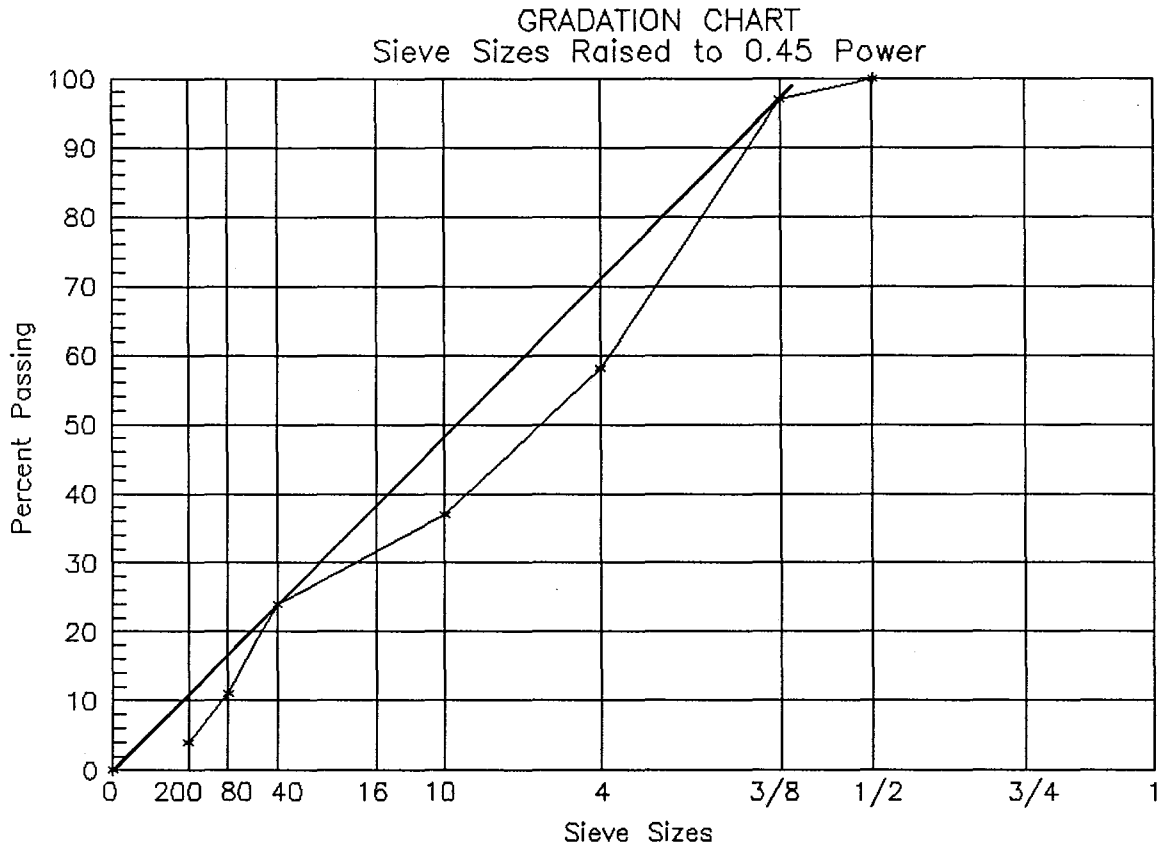


Figure 15. Mix Design and Extracted Gradations for Surface Mixture on Intersection Approach on SH 36 at Judge Ely in Abilene-District 8.

mixture properties are believed to have contributed to the rutting and, in some cases, shoving problems experienced by the failing intersection pavements:

1. The most common problem associated with intersection failure was plastic deformation manifested in the form of rutting. Ruts always became progressively deeper nearer the intersection. Shoving (usually manifested by transverse corrugations) was only a secondary problem at some locations. This combination of findings indicates that, the slower the traffic moves and the greater the frequency of horizontal forces (deceleration and acceleration) the greater the damage is to asphalt concrete pavement.
2. It appears that the leading materials related cause of intersection pavement failure was binder in the asphalt-aggregate mixture in excess of that required by the optimum mixture design.
3. Most of the mixtures studied contained relatively high percentages of natural (uncrushed) sand. The smooth, rounded, nonporous character of these fine aggregate causes the mixture to be sensitive to asphalt content and weak in shear strength, which thus imparts a higher propensity for permanent deformation. Approximately 30 percent minus number 40 sieve size material, which was largely field sand, was found in all the problem intersections. (State specifications for Item 340 Type D allow up to 40 percent passing the number 40 sieve.) Gap-graded mixtures containing rounded particles at the no. 40 sieve tend to be tender.
4. Aggregate gradations appeared to be very dense for some intersections that experienced early failure. Very dense aggregate gradations leave little room for asphalt binder; thus the mixture may become unstable with a

slight excess of asphalt.

5. Air void contents obtained from almost all the rutted intersection pavements were comparatively low (less than 3 percent), particularly in the wheelpaths. This indicates that either the mixture designs were too dense or that they were overcompacted during construction such that additional densification by traffic caused the mixtures to become unstable soon after construction and exhibit plastic flow (rutting and/or shoving).
6. The filler (minus #200) content of the paving mixtures was generally low (<4%). This condition also enhances sensitivity to binder content.
7. Many districts had established a two-year maintenance program where most intersection approaches in the district with significant traffic received treatment every other year.



RATIONAL APPROACH TO VERIFICATION OF MIXTURE SUITABILITY

THEORETICAL BACKGROUND

General

Scientific studies of hot mix asphalt concrete (HMAC) pavement design and performance have often been confined to the structural analysis of pavements without complete consideration of the role of the mixture variables that affect mixture strength. Current asphalt mixture design and pavement structural design procedures do not consider the influence of external (loading and structural) and internal (mixture) variables that alter mechanical response of the surface course to the applied loadings. They do not provide a direct relationship between distress mechanisms and fundamental mixture properties. The relationships that are presently available are based on empirical data from tests such as Marshall and Hveem procedures, and relationships between these "test" properties and performance criteria. Not only are these methods empirical in nature, they are not conducive to prediction of pavement performance when using mathematical models. These methods, however, have the advantages of being based on a large data base relating to pavement performance. A rational design procedure is needed to provide guidelines to indicate when a mixture, though adequate for certain applications, may not be suitable for a specific installation in a particular situation. Application of a rationally designed paving mixture can provide the most cost-effective approach to maximize pavement performance and/or minimize maintenance. Existing pavement design procedures do not include provisions for the design of approaches to intersections.

AASHTO Guide for the Design of Pavement Structures (1) provides guidelines and procedures which can be used for the determination of the total thickness of the pavement as well as the thickness of the individual layers comprising the pavement structure. These methods and most conventional mixture design procedures are based upon the observed behavior

of the pavements related to the thickness of the pavement layers. These design procedures are limited to the calculation of the needed pavement thickness to achieve certain pavement service life, which is an arbitrary value. Furthermore, since existing mixture design procedures are not based on proper evaluation of the stress and/or strain associated with the pavement loading and boundary conditions they are not suitable for use in the assessment and determination of the impending pavement distress, nor are they suitable for use in programming a rational plan for remedial action.

A proper mixture design requires complete assessment of the stress and/or strain state in situ, in concert with a complete description of the material characteristics (parameters that constitute pavement strength). Moreover, since pavement design engineers have to base their decisions on the conclusions of the experimental results obtained in the field and/or laboratory, it is essential that these parameters are determined with a testing method that best simulates field conditions and that the results correspond with the failure criterion selected to evaluate the distress mode under pertinent environmental conditions. This fundamental approach will subsequently ensure adequate pavement performance (response to loads).

Failure Criteria

The response of a pavement structure, and hence its failure, depends on the materials used, as well as the type and history of the applied loading. Accordingly, a suitable failure criterion must account for the influence of using different materials, different loading conditions, as well as other factors that affect the stress distribution within the pavement (such as type of base, interfacial bonding, etc.). Once the appropriate failure mechanism under the assumed service conditions is determined, a parameter such as stress, strain, or energy may be chosen as a critical or limiting parameter and used to evaluate the performance potential of the pavement structure (5).

A suitable test procedure must be adapted to determine the parameter deemed as critical to performance. It must be remembered that a single theory may not always apply to a given material, because the material may

behave in a ductile fashion under some conditions (hot climate) and in a brittle fashion under others (cold climate). A theory that works for ductile failure may not work for brittle fracture. Therefore, not only must the proper failure mode be defined, but also the various stress states likely to be produced within the pavement system must be considered. A virtually unlimited number of stress states are possible, but it is undesirable and even unacceptable to test at every one of them. In general, one is limited, for practical reasons, to test only a few specimens in order to obtain material properties. Thus, selection of the critical failure criterion is essential as this criterion defines at what point the material will fail in the selected distress mode and under the stress states expected to occur in the pavement system. This allows the presumption that the critical value of the parameter selected is achieved without regard to the stress state (5).

Situations exist in which permanent deformation in asphalt concrete pavement (ACP) occurs rapidly under relatively few load applications (6). This type failure of ACP is due to lack of stability in the mixture and thus the inability of the mixture to resist induced shear stress from wheel load applications usually resulting from improper mixture design or construction quality control (5).

It is possible for an asphalt concrete mixture to possess high tensile strength but lack sufficient internal friction (i.e., high percentage of hard asphalt). On the other hand, a good level of internal friction at higher temperatures does not ensure resistance to deformation when the confining pressure within the pavement layer is quite low (i.e., low stability and/or high temperature). In the latter case, the cohesive strength is the major contributor to shear strength (5).

Several theories are available for predicting failure of various types of materials. However, none of the theories agree with test data for all types of materials and combinations of loading. From the classic theories of failure, this research study considers octahedral shear stress to be the most appropriate criterion by which to examine the shear failure of ACP overlays resulting in permanent deformation early in the service life. Octahedral shear stress at failure for the ACP mixture will be defined by

the Mohr-Coulomb failure envelope. This is a valid approach because the stress and strain magnitude for a given material at a given point in a pavement structure is a direct function of the triaxial stress state (5).

Mohr-Coulomb Failure Theory

The application of the Mohr-Coulomb failure criterion is well documented in its application to soil mechanics. This criterion states that the failure of an isotropic material, either by fracture or by the onset of yielding, will occur when (in a three-dimensional state of stress) a Mohr's circle (having diameter $(\sigma_1 - \sigma_3)/2$ where $\sigma_1 > \sigma_2 > \sigma_3$) touches a failure envelope. This criterion may be used to predict the effect of a given state of stress at a point. The assumption is that the region enclosed by Mohr's circle for any possible state of stress not causing failure must be a region safe from failure. According to this criterion the shear strength increases with increased normal stress on the failure plane. Experimental evidence demonstrates that the envelope, which is tangent to all the failure circles and bounds the safe region, is usually slightly curved concave downward. A simple way to approximate the envelope is to draw a straight line tangent to at least two Mohr's circles.

Thus, a failure envelope is defined by Figure 16 and the Mohr-Coulomb equation:

$$\tau = C + \sigma_f \tan \phi \quad (1)$$

where:

- τ = shear strength, psi,
- σ_f = normal stress at failure, psi,
- ϕ = angle of internal friction, and
- C = inherent cohesive strength, psi.

In order to define the terms C and ϕ (cohesion and angle of internal friction, respectively), at least two triaxial tests must be performed: (1) an unconfined compression test and (2) a confined compression test with confining pressure that best simulates field conditions. Ideally, it is

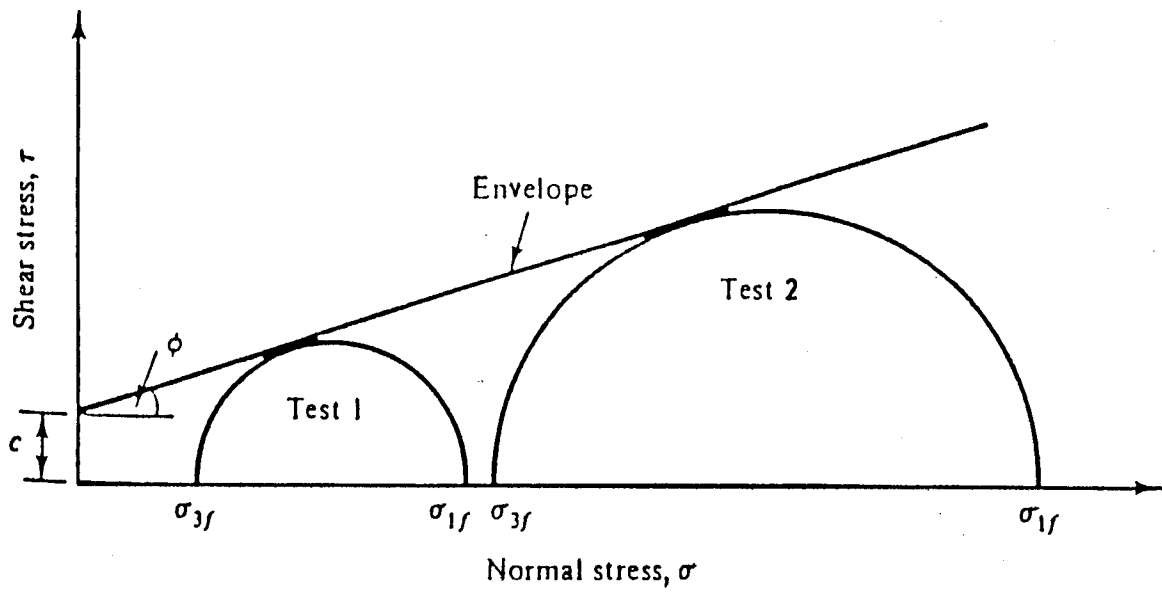


Figure 16. Parameters of Mohr-Coulomb Model Where σ_{1f} is the Major Principal Stress at Failure (After Reference 5).

preferred to conduct triaxial tests at several values of confining pressure. The values of these parameters (ϕ and C) could simply be determined as shown in Figure 16. This procedure is sensitive to the stress condition developed within the asphalt concrete, and the stress state can be defined adequately and relatively simply by the major (σ_1 or tire pressure) and the minor (σ_3 or confining pressure) principal stresses. With this method, the critically important conditions resulting from the tire pressure and interlayer bonding can be simply and accurately evaluated (7).

The Mohr-Coulomb failure envelope is a simple and direct method of evaluating stability of ACP overlays and their potential to resist rapid deformation. For a particular mixture, conditions causing failure under any vertical principal (compressive) stress from the wheel load can be calculated from the geometry of the failure envelope. The equation representing the relationship between major and minor principal stresses at failure is as follows (8):

$$\sigma_1 = \sigma_3 \left[\frac{1+\sin\phi}{1-\sin\phi} \right] + 2C \left[\frac{1+\sin\phi}{1-\sin\phi} \right]^{\frac{1}{2}} \quad (2)$$

where:

- σ_1, σ_3 = major and minor principal stresses,
- ϕ = angle of internal friction, and
- C = cohesion.

Equation (2) demonstrates that the maximum vertical stress that can be supported by any given material is influenced directly by lateral support, σ_3 , cohesion, C , and angle of internal friction, ϕ .

Octahedral Shear Stress Theory

Octahedral shear stress offers a scalar parameter which defines the influence of nine stresses at a specific point. This technique offers a method that is more directly quantifiable than the Mohr-Coulomb method.

Octahedral shear stress in a general form is defined as:

$$\tau_{\text{oct}} = \frac{1}{3} [(\sigma_x - \sigma_y)^2 + (\sigma_y - \sigma_z)^2 + (\sigma_z - \sigma_x)^2 + (\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2)]^{1/2} \quad (3)$$

where:

$$\begin{aligned} \sigma_x, \sigma_y, \sigma_z &= \text{normal stresses in x, y and z directions,} \\ \tau_{xy}, \tau_{yz}, \tau_{zx} &= \text{shearing stresses on xy, yz and zx planes, and} \\ \tau_{\text{oct}} &= \text{fundamental stress invariant.} \end{aligned}$$

Equation (3), in terms of principal stresses on a plane where shearing stresses are zero, will reduce to:

$$\tau_{\text{oct}} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2} \quad (4)$$

where:

$$\begin{aligned} \sigma_1 &= \text{major principal stress,} \\ \sigma_2 &= \text{intermediate principal stress, and} \\ \sigma_3 &= \text{minor principal stress.} \end{aligned}$$

Equation (2) can also be transformed to calculate the octahedral shear stress for any condition in an overlay structure (5):

$$\tau_{\text{oct}} = 0.942 \left\{ \frac{\sigma_3 \sin \phi}{1 - \sin \phi} + C \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^{\frac{1}{2}} \right\} \quad (5)$$

According to this theory, inelastic action at any point in the material under any combination of stresses begins when the maximum octahedral shear stress $(\tau_{\text{oct}})_{\text{max}}$ becomes equal to $0.471 \sigma_f$. This makes it possible to apply the strain energy of distortion criterion of failure, by dealing with

stresses rather than dealing with energy directly. Thus, failure occurs when $\tau_{oct} = 0.471 (\sigma_1 - \sigma_3)$, where $\sigma_2 = \sigma_3$.

Although the two theories (octahedral shear stress and Mohr-Coulomb) are completely different, a study of Mohr-Coulomb failure theory indicates that, at failure, the octahedral shear stress is exactly equal to 0.471 times the deviator stress. The ratio of actual octahedral shear stress in the pavement to the ultimate octahedral shear stress predicted by theory can be used to indicate how close to failure the overlay may be. The closer this value is to unity, the more likely it is that plastic deformation will develop at an accelerated rate.

Although the value of τ_{oct} may not be the maximum shear stress on any plane through a point in the paving mixture, it has the significance of being used to define the onset of yielding in a general state of stress.

Test Methods for Mixture Mechanical Properties

Mechanical properties of the paving mixtures are of great significance in their application to the pavement structure. When an asphalt mixture is subjected to external loads it behaves viscoelastically; that is, its deformation properties are both rate and temperature dependent and will exhibit both elastic and flow phenomena. Therefore, it is important that the testing method(s) selected to ascertain these properties adequately simulate the field conditions and clearly distinguish between elastic and time dependent properties of the mixture.

Marshall and Hveem are the two most widely used test methods to evaluate mixture stability. Other test procedures that are used to characterize mixture properties included static and dynamic creep, direct and indirect tension as well as unconfined compression tests. Each test method has certain unique features and some specific advantages. For example, a compression test is employed to determine the highest load that a compacted mixture of asphalt and aggregate can sustain at a preselected compressive load rate and temperature. It is possible in a one-parameter test, such as unconfined compression and indirect or direct tension, to vary the deformation rate and obtain some insight into the viscoelastic response

of the mixture. However, neither test provides sufficient information to determine the mixture's resistance to shear deformation. There is no way to determine the magnitude of the components (cohesive and frictional) that contribute to the layer bearing capacity. In a pure tensile test, no compressive stresses occur in the specimen, and in a compression test no tensile stresses are produced. A full understanding of the results obtained from these test methods is possible only if the missing information is acquired by means of other test procedures (9). Moreover, a one-parameter test does not account for the lateral support of the surrounding material. Hveem and Marshall test methods are empirical in nature and have no theoretical background to enable one to distinguish and separate the cohesive strength component from the frictional component of the total mixture strength.

None of the test methods mentioned above models the state of stress that causes yielding of the bituminous surface layer when subjected to horizontal and vertical surface loads.

It is generally agreed that the triaxial test is the most appropriate test by which to characterize the shear resistance and hence rutting resistance of particulate material such as asphalt concrete mixtures (10). Triaxially derived strength of the asphalt concrete mixture forms the basis for a "rational" method of evaluating plastic deformation potential and also more appropriately models the state of stress which exists in the pavement layers. The triaxial test measures two fundamental characteristics of bituminous paving mixtures: cohesion, C , and angle of internal friction, ϕ . The measured values of these parameters depend upon the temperature and the loading rate at which the testing is performed.

Typically, nominal high pavement temperatures are in the 120°F to 140°F range. Insofar as fast moving vehicles are concerned, bituminous pavements are subjected to loads of very short duration, and the viscous resistance developed by the bituminous mixture is quite high. Therefore, reasonably high loading rates for the laboratory testing of bituminous mixtures are justified for this type of traffic. However, for slow moving traffic, such as that at intersections, the time that a typical tire contact surface spends in contact with a point on the road surface has a significant effect

on the viscous resistance of the mix. In addition, many pavements on highways and city streets are subjected to vehicular braking and acceleration stresses that have severe effects on pavement performance and should be considered when designing laboratory test conditions. Cohesion and angle of internal friction are the two fundamental mixture parameters that have significant influence on the magnitude of the layer bearing capacity and must receive careful attention when measured in a laboratory triaxial test. A third resistance is also encountered in the mix when a paving mixture is subjected to traffic friction. This is the shearing resistance between the aggregate particles lubricated with asphalt which depends, among other factors, on the deformation speed. Nijboer (9) has presented an explicit discussion of how to measure and separate this strength component from the total mixture strength via a triaxial test with a complete examination of the validity of the assumptions regarding the triaxial testing procedure.

A recent survey study of tire pressure (11) in the state of Texas has shown that most trucks operate at a tire pressure of 98 psi or higher. Therefore, 100 psi may be considered to be a typical representation of tire pressure in Texas. If it is assumed, for a slowly moving vehicle (10 miles per hour) with such tire a pressure and an equivalent single wheel load (ESWL) of 9000 lbs, that the time that each element of a tire tread is in contact with the road surface is about 1 second, a 2-inch per minute stroke rate is justified for a paving mixture that exhibits a resilient modulus of about 100,000 psi (typical for a temperature near 100°F). This conclusion is based on a simple mathematical relationship which considers vehicle speed, tire pressure and, resilient modulus of a 4-inch deep ACP layer as follows:

$$V = \frac{ph}{tE} \quad (6)$$

where:

V = loading rate

p = tire pressure

h = thickness of the layer

E = modulus of resilient

$t = \frac{2r}{v}$, or time of contact, where

r = radius of circular tire print

v = vehicle speed

With this approach, one can employ the triaxial test at several values of confining pressure to investigate the resistance of a paving mixture and also predict the deformation rate at which a paving material will experience under service conditions. However, a major drawback with the triaxial test is that it is complex and time-consuming as it is currently employed.

A Simplified Procedure

Several researchers (12,13) endeavored to simplify the procedure by modifying the methodology. The straight line Mohr-Coulomb failure envelope can be defined simply and effectively by an indirect tension test and a direct compression test (unconfined compression) as shown in Figure 17 (12). The simplified procedure is capable of providing essentially the same fundamental materials characterization as the triaxial compression test, in significantly less time and with less sophisticated test equipment.

With regard to C and ϕ (cohesion and angle of internal friction) several researchers (9,10,14) have shown that the angle of internal friction is essentially independent of deformation rate and that it can safely be assumed to be isotropic (14); however, the magnitude of the cohesive strength parameter will vary with the deformation rate.

STRUCTURAL RESPONSE

General

As previously mentioned, the response (stress, strain, and deformation)

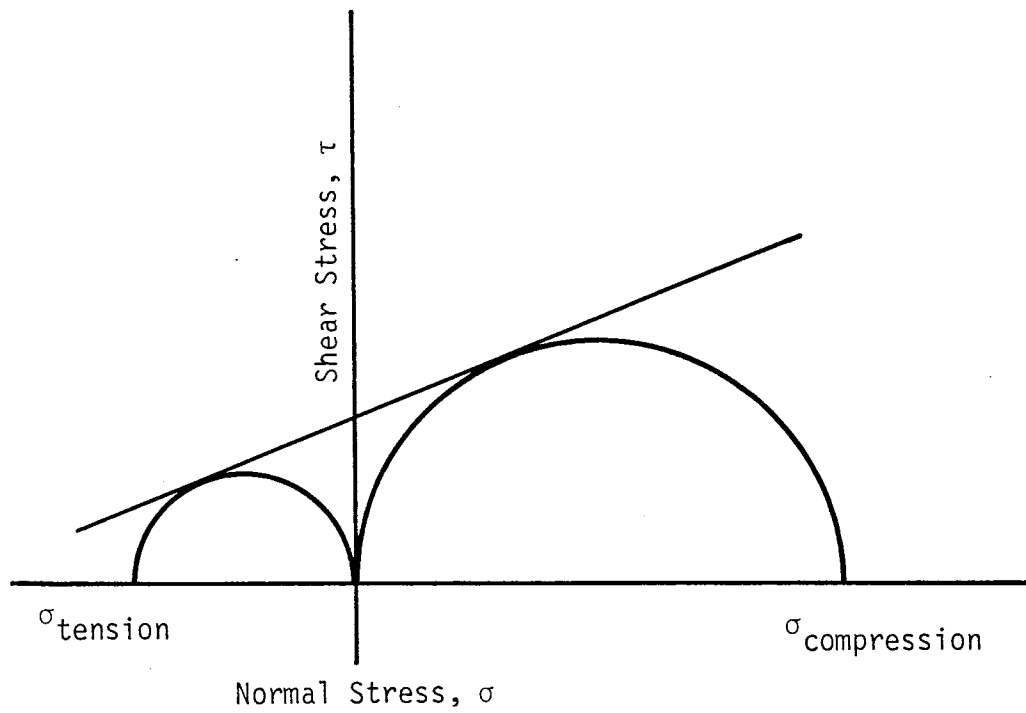


Figure 17. Mohr-Coulomb Failure Envelope Defined by an Indirect Tension Test and a Direct Compression Test.

characteristics of a paving mixture to the applied loads is not only elastic; it is also plastic, viscous, and viscoelastic/viscoplastic. The stress-strain relationship is often nonlinear and in many cases the material is anisotropic. To properly evaluate permanent deformation characteristics of an asphalt concrete pavement for a specific environmental and particular pavement boundary conditions, it is essential to accurately define the state of stress in the pavement structure.

Numerous sophisticated finite element computer programs have been developed to model three dimensional pavement structures and also to deal with some of the deviations from the assumptions of the classical theory of elasticity. However, almost identical stress distributions were obtained when the results from a nonlinear elastic half-space with a log-linear relationship between modulus and deviator stress were compared to the results from the solution of a linear elastic half-space structural model (15).

Although these computer programs are quite sophisticated, one should not assume that the results are exact. One method of verifying the accuracy of the responses obtained from any constitutive structural model is to compare the results to the actually measured field values. The extent of discrepancy will then dictate the extent of the modification required to the model and/or input parameters. It is only then that one can determine the validity of one analytical design method over another design procedure to predict pavement performance. Nevertheless, a general interpretation of the information from these models should lead to better understanding of the relative performance potential of designs for pavement structures.

The basic principle in the development of the traditional structural design methods has been to prevent excessive permanent deformation of the subgrade and to maintain the strain conditions in the wearing course and the base course within tolerable limits. However, not only is there not a well defined set of criteria for these parameters, but also these principles are accomplished by assuming that the vertical stress is a maximum directly under the wheel load at the surface, and that the horizontal radial strain is a maximum at the bottom of the surface layer directly under the applied load. Moreover, the traditional approach

neglects the influence of shearing forces at the surface produced under the wheel load rolling action. These assumptions will introduce significant errors when approaches to intersections are considered.

Claessen et al. (16) have shown that the maximum tensile strain does not occur at the bottom of the surface layer, nor does it always occur under the wheel load centerline. Its position depends upon the product of the modular ratio between the base layer and the surface layer (E_2/E_1), and the thickness of the asphalt layer h_1 .

Tielking et al. (17) have also shown that surface shear forces are developed when an inflated tire bears against a pavement surface as well as by the rolling resistance between the tire and the pavement surface. Horizontal loads are applied to the pavement surface when automobiles stop, turn, accelerate or decelerate. Restrained tangential motion due to vertical deflection of the tire generates tangential forces as shown in Figure 18 for both a stationary and a rolling tire.

Harrison et al. (18) have also shown that the influence of these horizontal shearing forces is more pronounced in the asphalt layer and are basically restricted to the upper part of the surface layer.

Octahedral Stresses

Ameri-Gaznon and Little (5) have performed a complete stress analysis of the surface course with and without the influence of horizontal surface shearing forces in combination with vertical load. In their study, they varied the level of interface bonding between the surface layer and the base layer for several values of surface thickness and stiffness placed on base pavement structures of both flexible stress sensitive materials and the portland cement concrete. They concluded that when shearing forces are applied in combination with the vertical load, the stress state is far more critical than that which has been assumed in conventional pavement design procedures. This demonstrates the importance of considering horizontal stresses in pavement design where intersections are concerned. They further showed that if slippage between the surface layer and the base layer occurs, the surface layer will act independent of the rest of the pavement system.

TIRE FORCES ON THE PAVEMENT

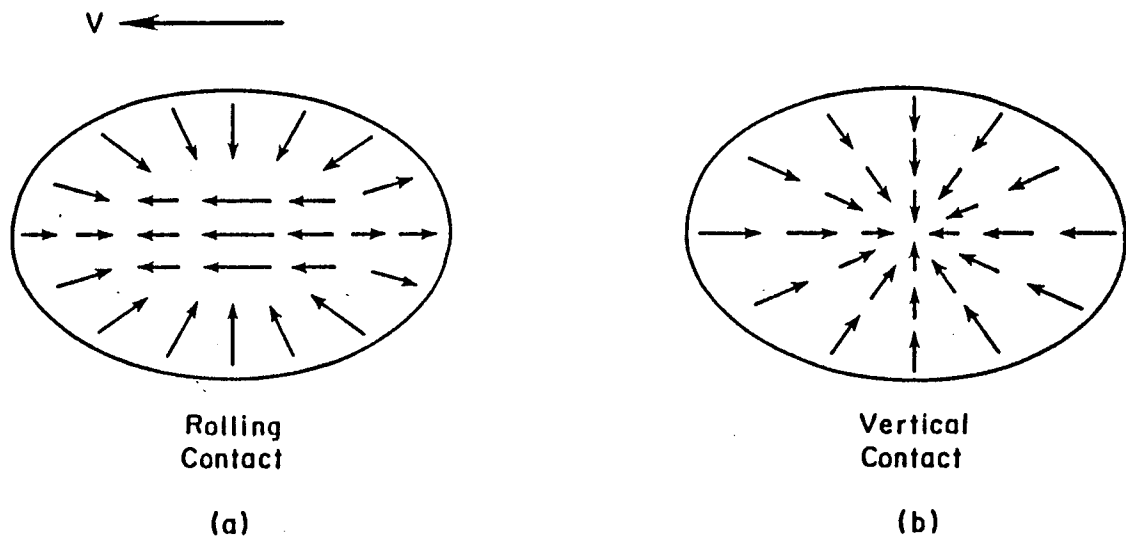


Figure 18. Distribution of Tire Shear Forces Under a Standing Tire (Vertical) Contact and Under a Rolling Tire (After Reference 17).

In this situation, the horizontal load must be completely withstood by the top, slipped layer. Their findings indicate that shearing stress has a more logical meaning in the study of stress state in a pavement than a vertical compressive surface load. This is true because, unlike vertical compressive stress, which is practically independent of the surface thickness, shearing stress distribution varies significantly with variation in the surface thickness. Moreover, plastic deformation is a shear failure and is a direct function of the triaxial stress field induced under applied loads.

With an appropriate computer program (such as modified ILLIPAVE) that allows incorporation of vertical load in combination with horizontal loads and includes provision for variations in the interface bonding between pavement layers, one can obtain reasonably reliable information to evaluate the stress state within the pavement structure under any loading and pavement boundary conditions. The corresponding, critical normal and shear stresses on the octahedral plane will depict the complete stress field and can be simply expressed in terms of stress invariants as follows:

$$\sigma_{\text{oct}} = 1/3 I_1 \quad (7)$$

$$\tau_{\text{oct}} = 1/3 (2I_1^2 - 6I_2)^{1/2} \quad (8)$$

where

$$I_1 = \sigma_x + \sigma_y + \sigma_z \quad (9)$$

$$I_2 = \sigma_x \sigma_y + \sigma_y \sigma_z + \sigma_z \sigma_x - (\tau_{xy}^2 + \tau_{yz}^2 + \tau_{zx}^2). \quad (10)$$

Quantities I_1 and I_2 are stress invariants because they are independent of how the coordinate axes are oriented in a given stress state. Furthermore, expressing octahedral normal and shearing stresses in terms of invariants does not require computation of principal stresses in order to determine normal and shear stresses on the octahedral plane.

Once the coordinate of the critical stress at a point in a given pavement structure is known, the required octahedral shear strength of the asphaltic layer can be characterized by a law similar to Mohr's strength law

proposed by Ameri-Gaznon and Little (5) as follows:

$$\tau_{\text{oct}} = C' + \sigma_{\text{oct}} \tan \phi' \quad (11)$$

where

ϕ' = transformed mixture frictional angle

C' = transformed mixture cohesive strength

This procedure is valid because, in the case of asphaltic materials, the first stress invariant must be taken into account for a particular stress state in which material strength is sought.

The ratio of actual octahedral shear strength in the pavement to the octahedral shear stress predicted by theory can be used to indicate how close to failure the layer may be. The closer this value is to unity, the more likely it is for the plastic deformation to develop at an accelerated rate.

Influence of Horizontal Loads and Interface Bonding

Evaluation of the bearing capacity of asphalt concrete mixtures has received considerable attention in the past three decades. Nonetheless, McLeod's approximate solution (19) is the only one to date which takes into consideration the influence of the frictional resistance between the tire and the pavement as well as the influence of the frictional resistance between pavement layers. McLeod's theoretical approach is similar to the calculation of the bearing capacity of a soil mass under static load in which the strength parameters C and ϕ (cohesion and angle of internal friction) are embodied into the solution where the plastic rupture mechanism is occurring.

In this solution, detrimental shear deformation in the upper layer is disallowed by assuming that the subgrade and the base course materials will not fail under the applied loads, and that detrimental plastic deformation of the subgrade is prevented by an adequate overall thickness of the base course and the wearing course. The materials selected for the base course

and the surface course layers also provide sufficient shear resistance to the stresses produced by the loads applied. It is worth noting that the bearing capacity calculation from the triaxial test procedure is based on the failure mechanism following a single cycle of loading. In actual practice however, the mechanism is developed as a result of repeated loading. Another limitation is that the critical strain may be reached under a load smaller than the maximum in which C and ϕ are determined.

A Corps of Engineers investigation (20) has indicated that continuous traffic on asphalt concrete pavements will ordinarily increase the density of the mix. In some cases, the air voids content will decrease below a critical value (2 to 3 percent by volume) and instability will develop. It is, therefore, advisable to obtain mixture strength parameters at a stress level somewhat smaller than the peak stress at which critical strain level is conservatively in concert with stress value. The triaxial test approach reduces the broad nature of the bearing capacity problem to measure the minimum values of C (cohesion) and ϕ (angle of internal friction) which correspond to the condition of failure.

McLeod's approximate solution for the stability of a bituminous concrete mixture based on the Mohr-Coulomb failure theory is given in Equation (12) which results from equilibrium at the critical state:

$$\sigma = 2C \left[\frac{K_p^{\frac{1}{2}} (2K \ell/w \tan\phi + J) + \ell/w + 1/K_p^{\frac{1}{2}}}{1/K_p + \ell/t(f-g)} \right] \quad (12)$$

where:

- σ = bearing capacity of the asphaltic layer (vertical stress),
- C, ϕ = as previously defined,
- K, J = coefficients expressing the influence of the vertical pressure outside the loaded area ($K=J=1$ is conservative),

$$K_p = \left[\frac{1 + \sin\phi}{1 - \sin\phi} \right]^{\frac{1}{2}} \quad (13)$$

- t = thickness of the surface layer,
 l,w = length and the width of the load area (tire footprint),
 f = coefficient of friction between tire and the pavement,
 and
 g = coefficient of friction at the surface and the base interface.

Equation (12) is based on the following assumptions:

1. Vertical load is uniformly distributed in both vertical and horizontal directions,
2. Rectangular contact area of the tire is proportional to the vertical load, and
3. Frictional resistance between tire and surface and interface bonding are also proportional to the vertical load.
4. Lateral support (σ_3) provided by the pavement immediately adjacent and surrounding the loaded area is equal to the unconfined compressive strength of the mixture expressed by Equation (14).

$$\sigma_3 = 2C \left[\frac{1 + \sin\phi}{1 - \sin\phi} \right]^{\frac{1}{2}} \quad (14)$$

In the situations where longitudinal cracks exist, the contribution to the bearing capacity by the pavement adjacent to the loaded area should be omitted by equating the following expression in Equation (12) to zero (21):

$$[(2Kl/w K_p \tan\phi^{1/2} + l/w] \quad (15)$$

The wall effect is defined as the effect of one of the four sides of the prism (with a rectangular base) that forms the tire model. Equation (12) neglects the tensile strength contribution to the asphalt bearing capacity provided by the rear wall due to the possible presence of transverse hairline cracks in the surface of the pavement.

The typical values of the parameters in Equation (12) are:

1. The value l/w is approximately equal to unity, although higher ratios can be used.
2. Coefficient of friction, f , between tire and the surface ranges from 0.30 to 0.8, depending on the type of road and tire condition. A value of 0.8 is typically used for locked-wheel braking on dry pavement; however, for an unlocked, slow-moving tire on dry pavement, f may approach 1.0.
3. The value of the coefficient of interface friction, g , is dependent on the asphalt overlay temperature and the magnitude of vertical pressure, as well as the tack coat rate. No specific value or range of values are reported in the literature regarding this variable. However, the maximum value that g can attain is equal to unity.

Examination of Equation (12) indicates that when the frictional resistance between the pavement and tire is equal to the maximum frictional resistance between the pavement surface and the pavement base (i.e., $f - g = 0$) (Figures 19 through 22), thickness of the pavement has no influence on the bearing capacity which is developed by the pavement when subjected to severe braking or accelerating stresses. Moreover, when the frictional resistance between the pavement and the tire is less than the maximum frictional resistance between the pavement surface and the pavement base ($f < g$), an increase in pavement thickness leads to a decrease in the bearing capacity developed by the pavement under braking stresses. Consequently, the influence of overlay thickness on bearing capacity is a function of the $(f-g)$ value.

The following items should be noted from Figures 19 through 25:

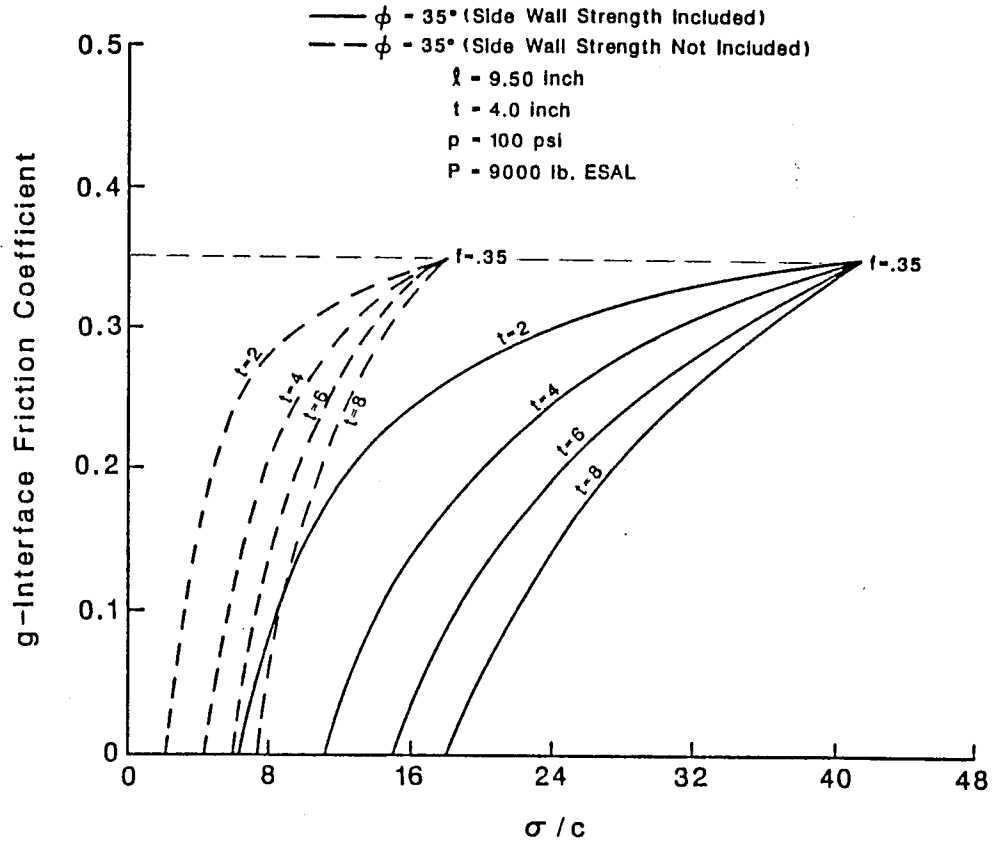


Figure 19. Bearing strength as a function of interface bonding for constant friction ($\phi = 35^\circ$, $f = 0.35$) (After Reference 5).

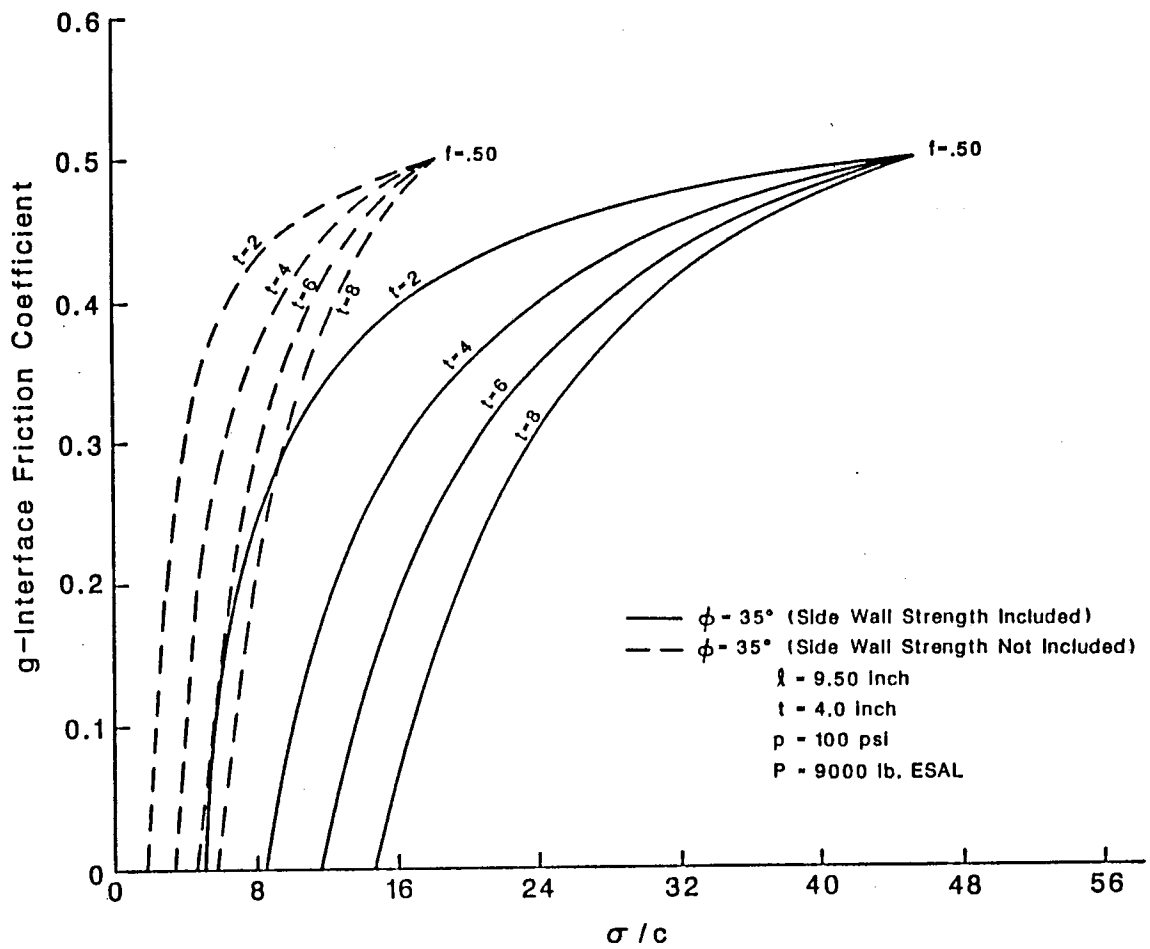


Figure 20. Bearing strength as a function of interface bonding for constant friction ($\phi = 35^\circ$, $f = 0.50$) (After Reference 5).

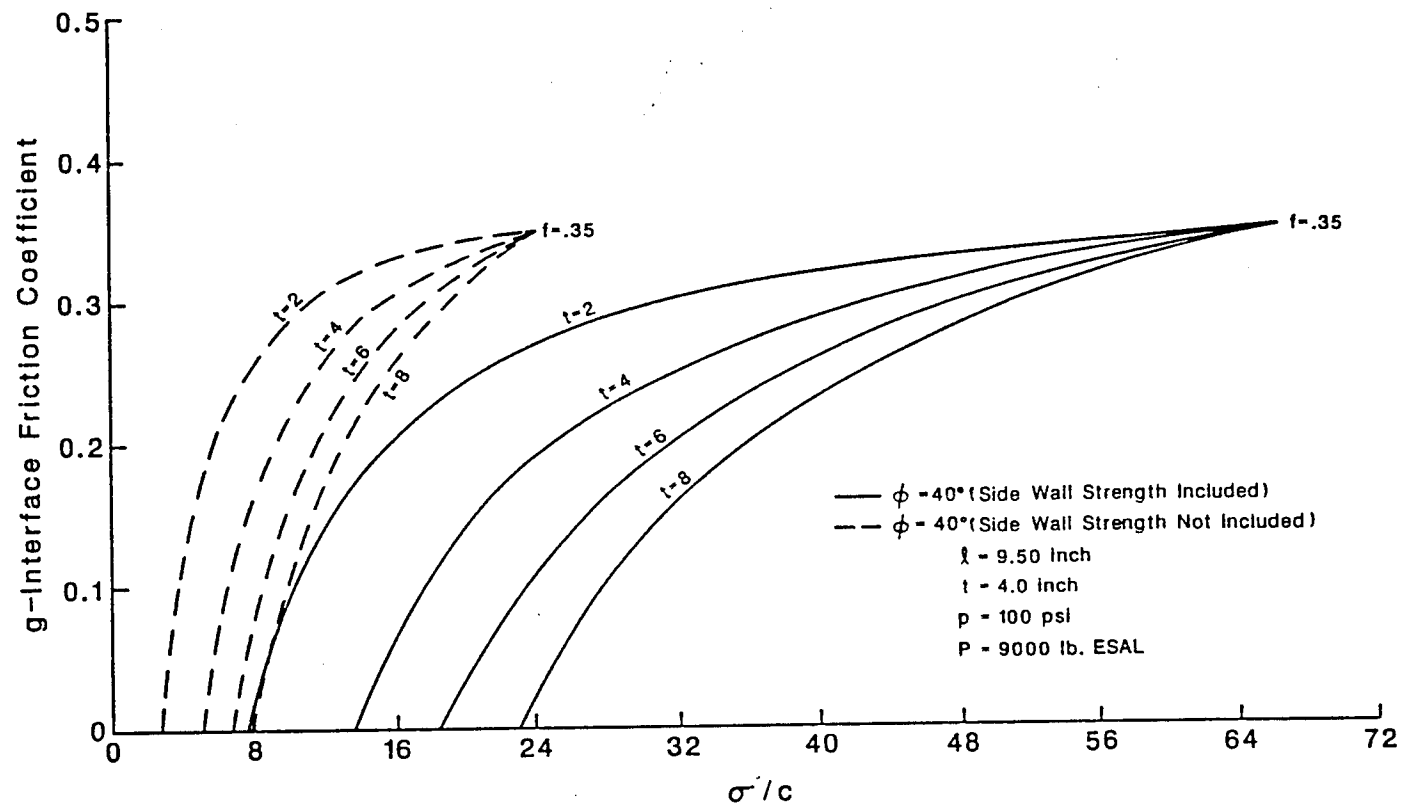


Figure 21. Bearing strength as a function of interface bonding for constant friction ($\phi = 40^\circ$, $f = 0.35$) (After Reference 5).

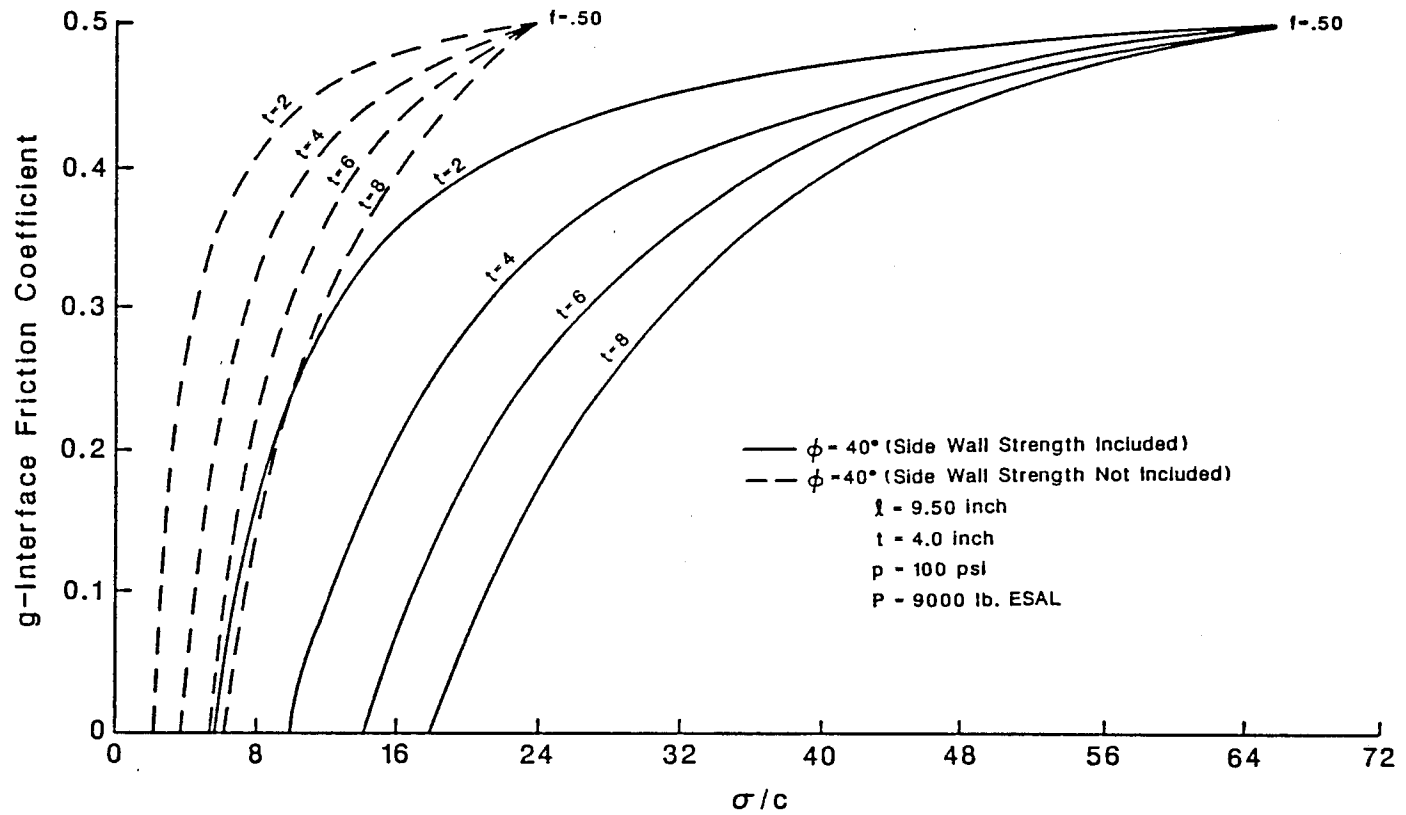


Figure 22. Bearing strength as a function of interface bonding for constant friction ($\phi = 40^\circ$, $f = 0.50$) (After Reference 5).

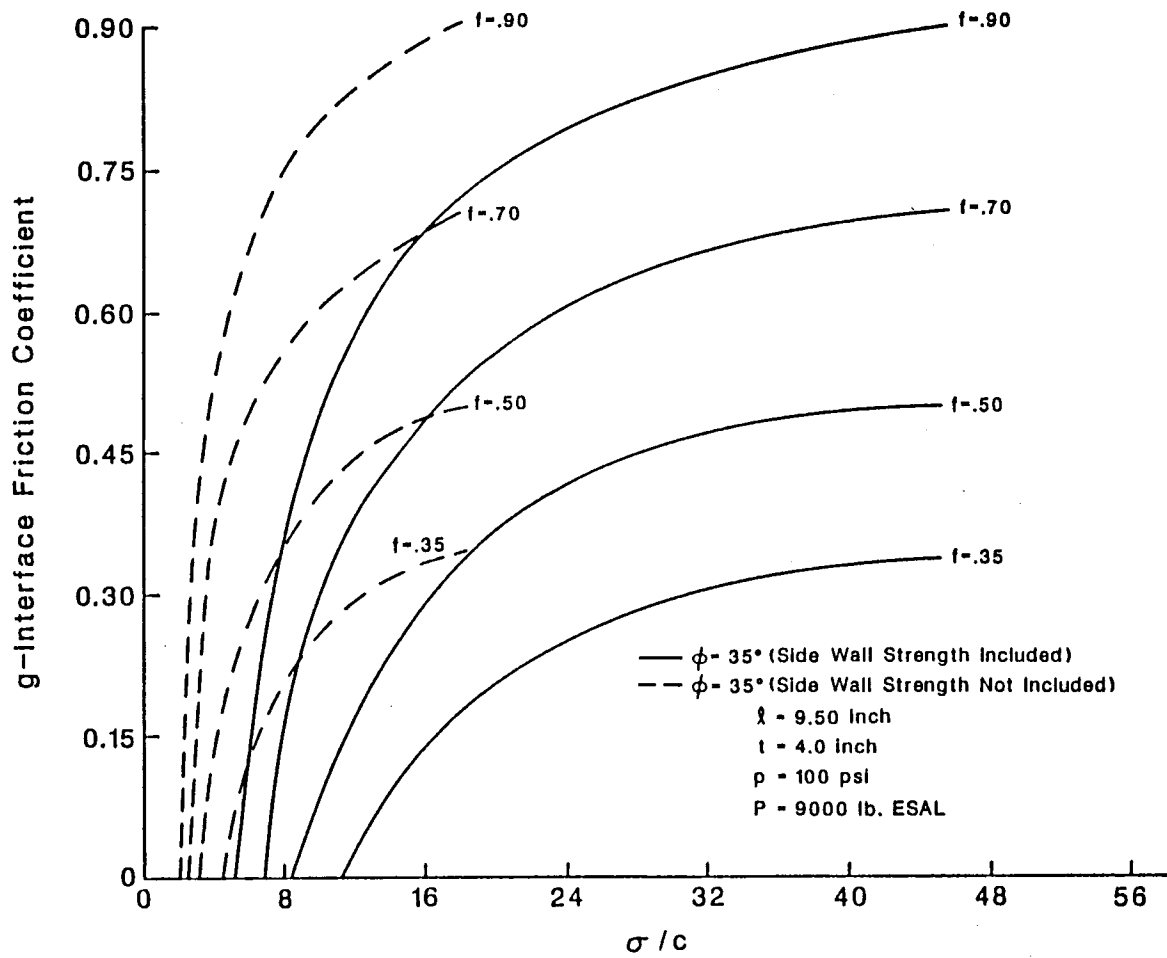


Figure 23. Bearing strength as a function of interface bonding for various levels of surface friction ($\phi = 35^\circ$, $f = 0.35 - 0.90$) (After Reference 5).

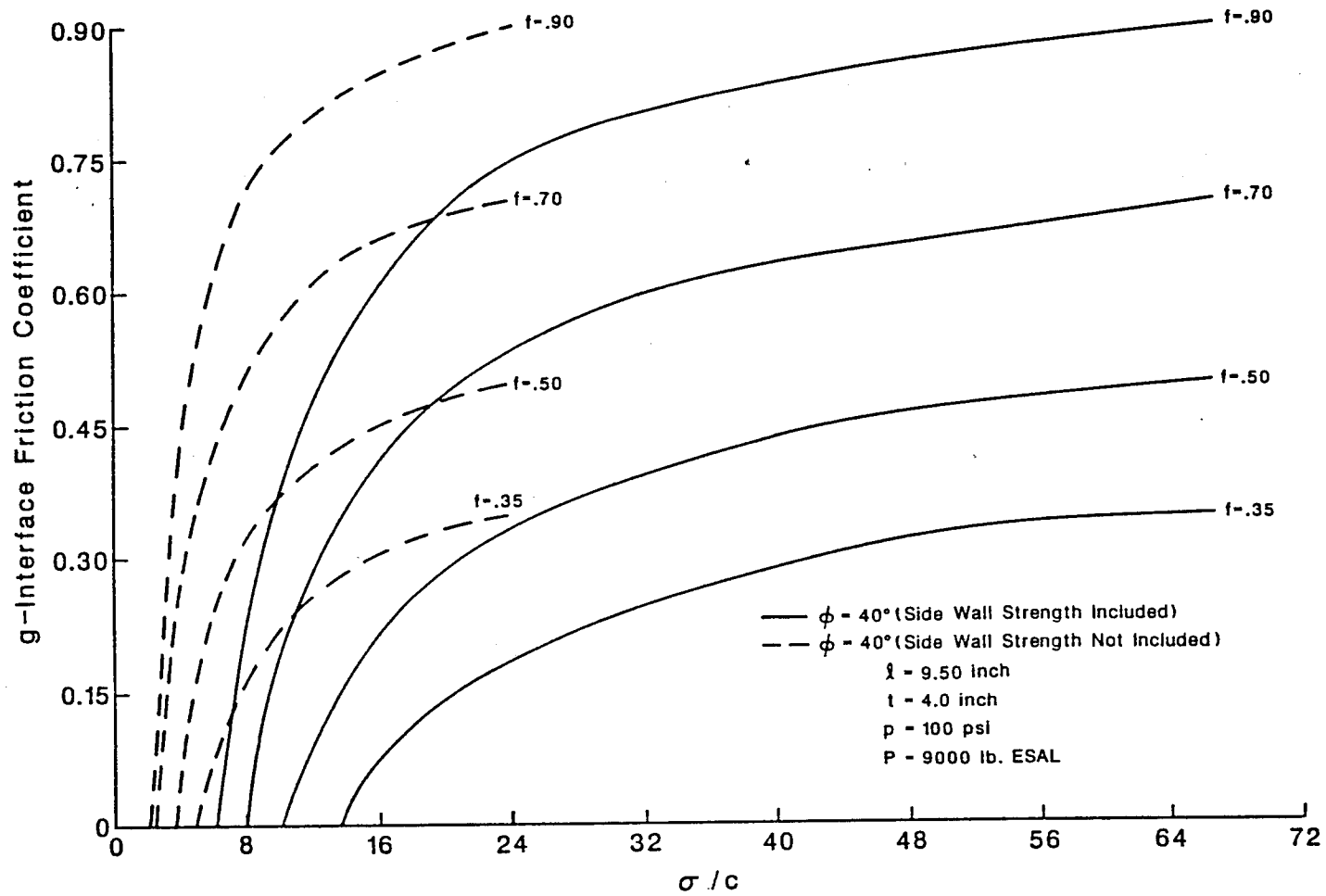


Figure 24. Bearing strength as a function of interface bonding for various levels of surface friction ($\phi = 40^\circ$, $f = 0.35 - 0.90$) (After Reference 5).

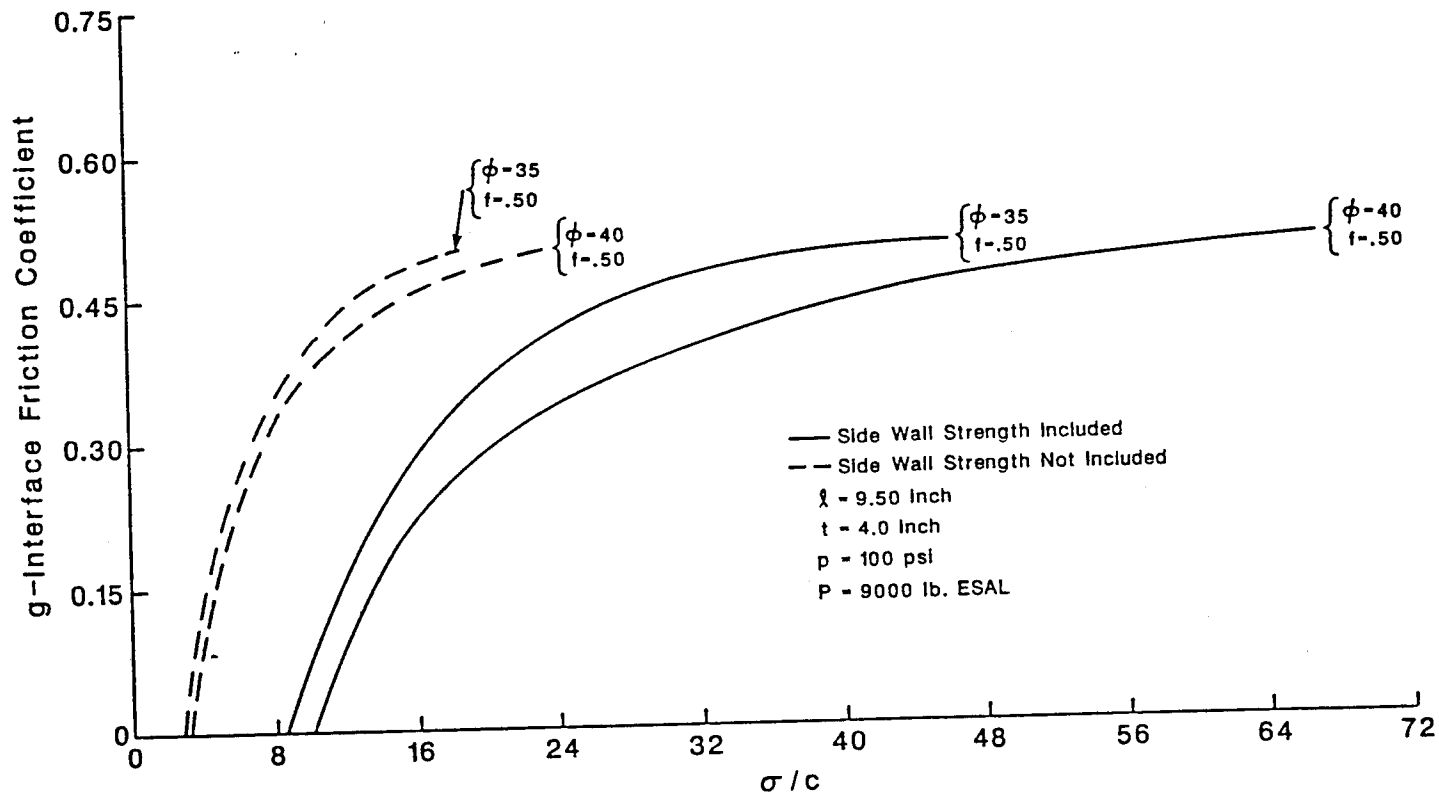


Figure 25. Bearing strength for different values of mixture internal friction and overlay surface friction (After Reference 5).

1. When frictional resistance at the interface between the surface layer and the base layer is equal to the frictional resistance to sliding between the tire and the surface, increasing surface thickness will not contribute to the bearing resistance of the pavement. In this case, the main contributor to the pavement bearing strength is the cohesive strength of the mixture.
2. When frictional resistance at the interface between the surface layer and the base layer is less than that between the tire and pavement, an increase in surface thickness will increase the bearing strength of the pavement.
3. The contribution to the pavement bearing strength provided by the side walls of the theoretical rectangular tire footprint, immediately surrounding the loaded area, is more than half the layer bearing strength.

The most critical condition in a pavement bearing capacity analysis is when high frictional resistance exists between the tire and the pavement surface and when there is little or no frictional resistance at the interface between the surface layer and the base layer space. Therefore, in any bearing capacity analysis for moving traffic, one must include the horizontal surface force in the analysis and consider the combined effects of horizontal and vertical stresses. In addition, static loads are critical for design because of creep within the ACP.

The frictional forces that are developed at the pavement-tire interface during vehicle maneuvering is itself a complicated phenomenon that is not yet fully understood. However, the magnitude of the horizontal surface force can be calculated by elementary laws of mechanics first reported by Coulomb. Although Coulomb's solution is not exact, its application provides reasonably accurate information to formulate a simple model to calculate shear forces at the pavement surface. In Coulomb's solution, the magnitude

of horizontal force is dependent upon the frictional resistance between tire and the pavement, which vary with pavement and tire conditions as well as tire structure and tire print geometry. A coefficient of friction of 0.90 or more is not uncommon for a dry pavement subjected to a braking force (wheel not locked) of a slow moving vehicle, as in an intersection approach.

The modified ILLIPAVE computer program allows incorporation of horizontal surface forces in addition to the vertical loads at the surface and provides a complete illustration of stress state throughout the pavement layers. This program can be used to calculate the maximum (critical) stresses in the pavement layers due to the surface loads.

RUT DEPTH ASSESSMENT

Bituminous materials are viscoelastic and their stress-strain characteristics are dependent upon both rate of loading and temperature. Because of the significant effects of these variables, Van der Poel (22) introduced the term stiffness modulus in which the stress-strain characteristics of bound bituminous materials are expressed as a function of the loading time, t, and temperature, T, as follows:

$$S_{(t,T)} = \left[\frac{\sigma}{\epsilon} \right]_{(t,T)} = \frac{\text{stress}}{\text{total strain}} \quad (16)$$

It should be noted that this modulus is not the same as the conventional elastic modulus. Figure 26 (22) illustrates the variation of this stiffness modulus with time and temperature for a typical asphaltic mixture for a base course. Under usual traffic conditions, the stiffness modulus of an asphalt mixture may vary from about 2×10^3 psi at high temperature and creep speed to about 3×10^6 psi at low temperature and fast speed. Therefore, the desired value of the mixture stiffness is obtained only if the measurements are made at the appropriate time, temperature, and stress conditions (22). In addition to the above influential factors, the mixture stiffness is also dependent upon the mixture specifics which include:

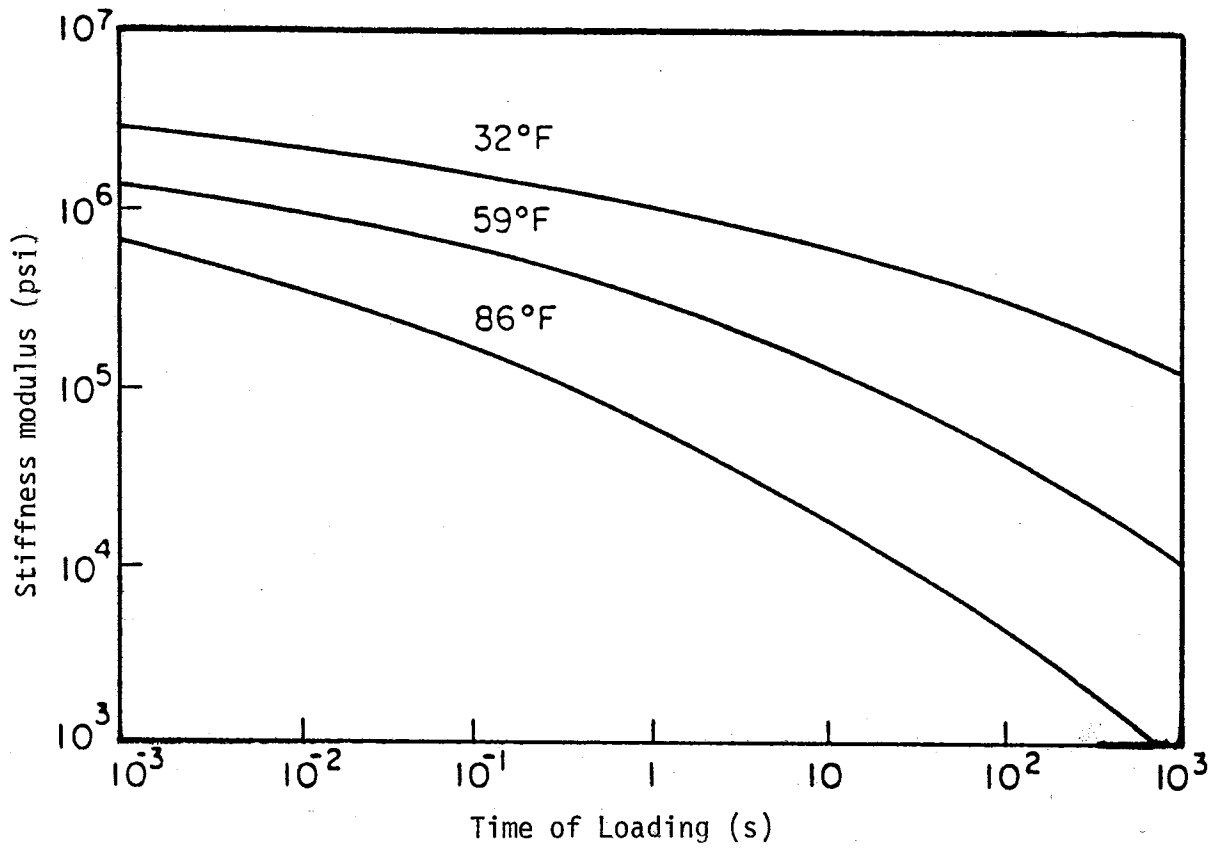


Figure 26. Effect of Loading Time and Temperature on the Stiffness of a Typical Asphalt Mix for a Base Course (After Reference 22).

1. Aggregate type (angularity, surface texture)
2. Aggregate grading
3. Asphalt grade (viscosity)
4. Asphalt content, and
5. Air voids

One method of estimating mixture stiffness is to use the method developed by Shell researchers as follows:

$$S_{mix} = S_{bit} \left[1 + \left(\frac{2.5}{n} \right) \left(\frac{C_v}{1 - C_v} \right) \right]^n \quad (17)$$

where:

S_{mix} = stiffness of the asphalt concrete mixture (kg/cm²) and

S_{bit} = stiffness of bituminous binder at the desired temperature and time of loading.

C_v = volume concentration of aggregate in the mixture

$$C_v = \frac{V_{aggregate}}{V_{aggregate} + V_{asphalt}}$$

$$\begin{aligned} n &= 0.83 \log \left(\frac{400,000 \text{ MPa}}{S_{bit}} \right) \\ &= 0.83 \log \left(\frac{5.8 \times 10^6 \text{ psi}}{S_{bit}} \right) \end{aligned}$$

Equation (17) is applicable for a mixture with C_v values between 0.7 and 0.9 and air void contents of about 3 percent. For mixtures having air voids greater than 3 percent, the volume concentration of aggregate should be corrected by C'_v as follows (22):

$$C'_v = \frac{C_v}{1 + H} \quad (18)$$

where:

H = the difference between the actual air void content and 3 percent expressed as a decimal.

It should also be pointed out that the above correction is applicable only to mixtures with a volume concentration that will satisfy the following expression:

$$C_B > \frac{2}{3} (1 - C'_v) \quad (19)$$

where:

$$C_B = \frac{V_{\text{asphalt}}}{V_{\text{aggregate}} + V_{\text{asphalt}}}$$

In Equation (17), S_{bit} can be read directly from the nomograph (Appendix A) and/or be obtained from the following expressions within the Vander Poel limits specified:

$$S_{\text{bit}} = 1.157 \times 10^{-7} \times t_w^{-0.368} \times \exp(-PI) \times (T_{\text{RB}} - T)^5 \quad (20)$$

where:

- S_{bit} = bituminous stiffness in MPa,
- t_w = loading time in seconds,
- PI = penetration index,
- T_{RB} = ring and ball test temperature, °C, and
- T = temperature of the bitumen, °C.

With bitumen stiffness in psi and temperature in degrees Fahrenheit Equation (20) becomes:

$$S_{\text{bit}} = 8.878 \times 10^{-7} \times t_w^{-0.368} \exp(PI) \times (T_{\text{RB}} - T)^5 \quad (21)$$

The relationship will give an approximate value of the bituminous stiffness within the following limits:

$$0.01 \text{ sec} < t_w < 0.1 \text{ sec}$$

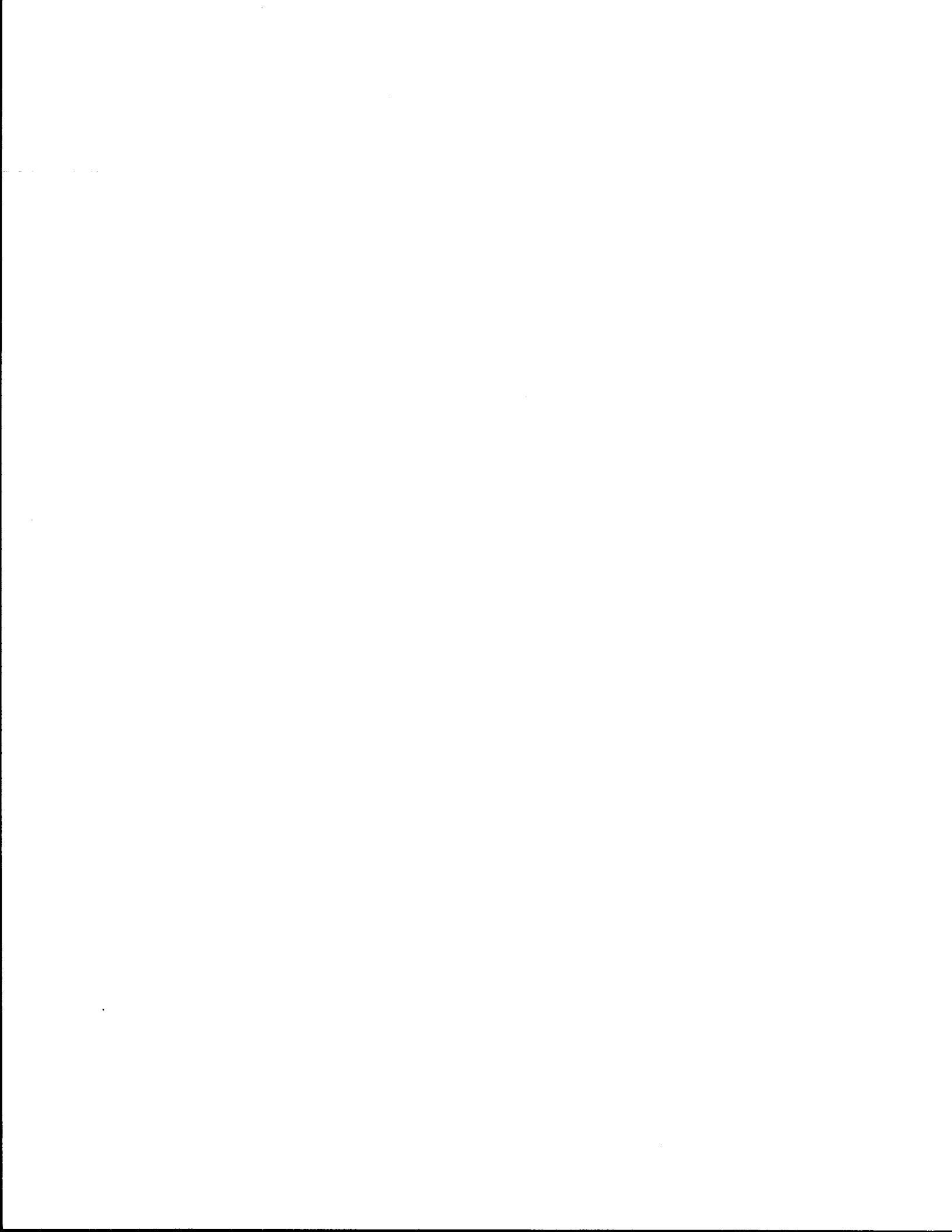
$$- 1 < PI < + 1$$

$$10^\circ\text{C} (50^\circ\text{F}) < T_{RB} - T < 70^\circ\text{C} (160^\circ\text{F}).$$

Once the stiffness of the mix is obtained from Equation (17) or any other reliable laboratory procedure, the critical strain in the surface course is obtained as follows:

$$\epsilon_{\text{critical}} = \frac{\sigma}{S_{\text{mix}}} \quad (22)$$

where σ represents the bearing strength of the surface course obtained from Equation (12).



MIXTURE VARIABLE ANALYSIS

Fundamental analysis of the mixture variables is possible by describing the asphalt-aggregate system in terms of the components of which it is comprised. That is, to divide the system into the constituent phase, and investigate the role of each variable and its function with respect to the failure mechanism of interest (rutting, cracking, etc.). The asphalt-aggregate system consists of three components, i.e:

1. solids (aggregate),
2. liquid (bitumen), and
3. gas (air)

In asphalt paving mixtures, aggregate usually comprises between 90 to 95 percent of the weight and between 80 to 85 percent of the volume of the mixtures. Moreover, the aggregate is primarily responsible for the load supporting capacity of the asphalt mixtures. Therefore, particular attention must be given to the physical properties of aggregate which include:

1. aggregate grading,
2. particle size,
3. particle shape, and
4. particle surface texture

The primary step in development of a mixture design is to select a set of aggregates that can be combined to meet the specification requirements. Characteristics of the aggregates and their specific gradations will depend on the available materials and preferences of the asphalt pavement designer. Of course, gradation bands are used as guides in the initial aggregate selection process (2,23). Because the aggregate is the largest constituent in an asphalt concrete mixture, the overall characteristics of the mix are largely dependent on the characteristics of the aggregate used. Such aggregate factors as quality, economy, durability,

strength, size, shape, surface texture, permeability, and gradation of the aggregate influence the final characteristics of the mixture. Selection and gradation of aggregates may be varied by using special specifications or provisions to meet specific job requirements which are related to economy, traffic demand, subgrade and environmental conditions.

AGGREGATE GRADATION

The gradation of aggregates from coarse to fine will give significant insight in understanding the role of the aggregates in asphalt concrete. Gradation will affect such properties as stability, density, workability, and permeability (durability and moisture susceptibility).

Herrin and Goetz (24) studied the effect of aggregate shape on the stability of both gap-graded and dense-graded mixtures. They found that changing the amount of crushed coarse aggregate from zero to 100 percent does not appreciably affect the cohesive strength of the mix nor does it influence the magnitude of frictional component of the mix. However, regardless of the type of coarse aggregate used, the cohesive strength of both gap-graded and dense-graded mixture were significantly improved when the fine aggregate was changed from a rounded, smooth-texture sand to angular, crushed limestone particles. Button and Perdomo (25) obtained similar improvements in resistance to creep and permanent deformation when natural sand was replaced with crushed limestone particles of the same gradation. They demonstrated that Hveem and Marshall stability exhibited a greater decrease for a given increase in asphalt content above optimum for mixtures containing the higher percentages of natural sand particles.

Generally, the influence of mineral filler is two-fold (26), (1) filler occupies the voids between larger particles and provides more contact points (i.e., reduced aggregate contact pressure), and (2) when mixed with asphalt cement, the two form a high consistency matrix (mastic) which cements the larger aggregate particles together.

Although the compaction method greatly influences the final air voids content in the mix, adequate voids in mineral aggregate (VMA) must be obtained by adjustment in the aggregate grading.

SURFACE TEXTURE

Literature abounds on the influence of aggregate surface texture on the mixture stability. In general, the optimum asphalt content of a mix made using rough surface aggregates is higher than a mix of similar gradation made using smooth surface aggregates (27). This is primarily due to the increased surface area per unit weight on the rougher aggregate. Comparison of the mixes made with smooth-textured aggregate with those mixtures made with rough-textured aggregate of the same gradation and equivalent asphalt content revealed that mixtures containing rough aggregate exhibit greater strength and stiffness (27) and, particularly, resistance to permanent deformation (25,28). Rough surface aggregate will also enhance resistance to stripping and provide better skid resistance.

PARTICLE SHAPE

The shape of aggregate particles has appreciable effects on the physical properties of the paving mixture. Angular aggregate will require slightly more asphalt and provide greater VMA than rounded materials. The generally accepted principle that the shape of the coarse aggregate is critical with regard to adequate mixture properties seems to apply only to open-graded mixtures. The bulk of the literature indicates that the characteristics of the fine aggregate fraction are dominant for dense-graded mixtures (28 through 35). Following his study of pavement distress in intersections, Kandhal (36) recommended the use of angular fine aggregate (manufactured sand) to improve the creep behavior of the asphalt mixture. More angular aggregate will increase stability, resistance to plastic deformation, required compactive effort, and skid resistance, and will decrease workability.

MAXIMUM PARTICLE SIZE

It is well known that mixtures made with larger size aggregates exhibit a greater stability or resistance to shear displacement than similar mixes

made with a smaller aggregate size. The larger the aggregate size, the smaller the surface area per unit weight or volume of aggregate. Therefore, increasing the top-size of the aggregate will generally decrease the optimum void content and the optimum asphalt content. Larger aggregate also require less energy to produce. Consequently, asphalt mixtures made using larger size aggregate are more economical to produce.

The problems with the use of larger size stones in a mix lie in construction. It is more difficult to place larger stone sizes because they reduce paver performance and tend toward segregation. Also, mixes made with larger stone size require higher levels of compaction to achieve required mixture density.

Larger size aggregates were popular from the turn of the century through the 1950's (37). Pavements placed in the early 1900's were characterized by larger top size stone, high volume concentration of aggregate and low air voids. These pavements gave excellent service for over fifty years. However, their use was abandoned since finer stone sizes were easier to handle and did not wear the flights in the drum mix facility as much. There is recent evidence, however, that large stone mixes are making a comeback across the country (38 through 41) including District 1 in Texas (42).

BINDER CONTENT

Binder content is a compromise in which the final product must strike a favorable balance between the stability and durability requirements for the intended use (2). Durability of the asphalt concrete mixture is primarily a function of air voids content which is controlled among other things by asphalt content. Thicker films of asphalt binder will reduce the pore size of interconnected voids in the mix which makes it more difficult for the air and the water to penetrate into the layer. A limited laboratory study of the influence of binder content on mixture stability and durability indicated that increasing the quantity of asphalt binder by 0.5 percent above the optimum will increase pavement durability significantly (43); however, this benefit is achieved at the expense of a significant

decrease in mixture resistance to shear deformation (44). It should be pointed out that a mix containing a high percentage (>15 percent) of natural sand, small top-size aggregate (<½-inch) and/or low filler content (<4 percent) will likely be quite sensitive to binder content and will become unstable with a slight excess of asphalt. With these type mixtures, binder content must be carefully controlled at the mixing plant to avoid disaster.

BINDER GRADE (VISCOSITY)

Asphalt concrete mixtures containing a harder grade of asphalt (more viscous binder) will usually exhibit higher elastic stiffnesses which may result in more resistance to shear deformation. The stiffer mixture, however, is also more susceptible to cracking due to low temperatures and heavy loads over flexible substrates. In order to gain the advantage of high binder viscosity at high temperatures (to reduce plastic deformation) and not adversely affect mixture properties at low temperatures, one should consider asphalt modification, using one of several commercially available polymers.

Increasing binder viscosity should be considered only when minor improvements in resistance to shoving and rutting are needed. Major improvements should be addressed by improving aggregate quality. If well graded, high quality aggregates with sharp edges and good surface friction are used in a mix, the grade of asphalt cement plays a relatively small role in the rut resistance of a mixture (45 through 47).

AIR VOIDS

Air void content of a paving mixture will directly affect tensile properties and permeability. Permeability, of course, directly affects durability and water susceptibility. Dense graded mixtures with excessive air voids will usually exhibit failure mechanisms such as raveling, stripping or cracking. Low air voids in dense graded mixes may cause failure due to permanent deformation or flushing. Low air voids may be due to too dense an aggregate gradation and/or excessive asphalt. The key to

building deformation resistant asphalt pavements is to design and build a structure where aggregate particles carry the load and asphalt cement waterproofs and binds. Voids criteria is critical to the success of this kind of pavement. Enough voids in the mineral aggregate (VMA) must be present to provide room for asphalt cement and air voids. Adequate air voids are necessary to prevent pressure buildup of asphalt within the binder portion of the pavement (46). Kandhal (36) in his intersection study, found that when the density of an asphalt pavement approaches the theoretical maximum (minimum possible VMA) and when the mix could no longer consolidate, it rapidly lost stability and began to rut and shove. He concluded that durability and resistance to permanent deformation of mixtures can be maximized by increasing the VMA of the mix by deliberately deviating from the maximum density line for gradation (preferably towards the coarser side) and by increasing the design air voids in the mix.

APPLICATION OF FINDINGS TO INTERSECTION ENGINEERING AND CONSTRUCTION

GENERAL

Although design engineers have no control of the traffic volume, traffic loads, and/or environmental factors, adequate construction quality control as well as properly designed paving mixtures and structural systems are well within their jurisdiction. A well designed asphalt paving mixture that is correctly mixed and placed can withstand the shear and compressive stresses of heavy traffic at intersection approaches and will exhibit adequate resistance to deformation when temperatures and wheel loads are at the peak.

Button and Perdomo (25) in their investigation of rutting in asphalt concrete pavements in the state of Texas have identified several factors that have contributed to the pavement rutting. These factors are directly related to materials specifications, construction quality control and mixture design parameters. Excessive binder content, high percentages of fine (sand size) aggregate, and the rounded shape and smooth texture of fine aggregate particles are chief factors that have materially contributed to the deformation potential of the pavements.

In this study of hot mix asphalt concrete intersection pavements, those same factors were found to be the leading causes of premature failure. The mixture deficiency found to occur most often in the field experiments described earlier was excessive asphalt content. Apparently, asphalt content is difficult to control by the contractor and/or the upper allowable limit is sometimes increased by the inspector in order to achieve the required density. High percentages of natural sand, the glassy character of most sand particles, small maximum size aggregate, and relatively low filler contents tend to produce binder sensitive mixes: that is, these mixtures depend a great deal on the asphalt cement for shear strength. Since excess asphalt must be dealt with on occasion, mixtures with minimum sensitivity to asphalt content should be required.

The following paragraphs offer suggestions designed to provide a margin of safety to minimize premature failures of specially stressed intersection pavements.

HOT MIX ASPHALT CONCRETE SPECIFICATIONS

Texas SDHPT specifications for Item 340, hot mix asphaltic concrete pavement, allow and possibly encourage the use of gap-graded mixes. These mixtures are characterized by the "hump" in the gradation curve near the number 40 sieve and a relatively flat slope between the number 40 and the number 10 sieve. This indicates a deficiency of material in the number 40 to number 8 sieve size range and an excess of material passing the number 40 sieve (48). Mixtures of this type, particularly when the fines are composed primarily of natural sand, are termed "critical" in that they lack resistance to plastic deformation, tend to rapidly lose stability if the asphalt content exceeds optimum, and become tender and shove during periods of hot weather. One method of improving the aggregate grading specification to yield tough intersection mixes would be to lower the upper limit of the total percentage of material allowed to pass the number 40 and 80 sieves. An example of this suggested specification change is shown in Figures 27 and 28 for Item 340, Type C and D, respectively. According to Chastain and Burke (49), in 1957, less than 20 percent of highway agencies allowed more than 37 percent passing the number 40 sieve and more than 40 percent of them required less than 32 percent passing the number 40 sieve. Additionally, the existing Type C specification does not allow a sufficient quantity of large aggregate to effect their mutual interlock.

Specifications for Item 340 present gradation requirements in terms of percent passing and retained on selected sieves with an additional requirement for the total percent retained on the number 10 sieve. It is difficult to accurately convert the specification limits to total percent passing so that they can readily be plotted on a gradation chart. Changing the mixture gradation specification to total percent passing would facilitate plotting on standard gradation charts, yield a better

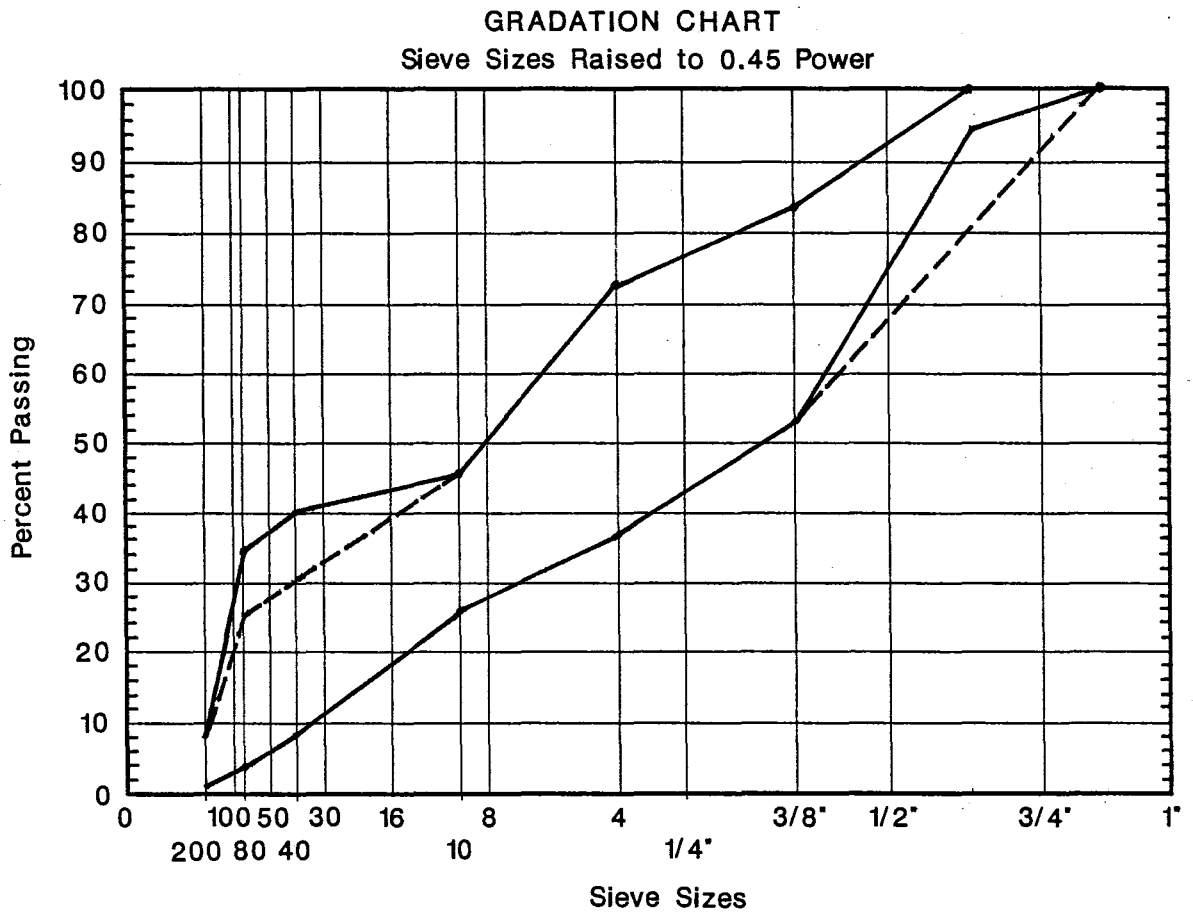


Figure 27. Aggregate Specification Limits for Item 340, Type C with Suggested Modification (dashed line) to Provide Tougher Mixtures for Intersection Approaches.

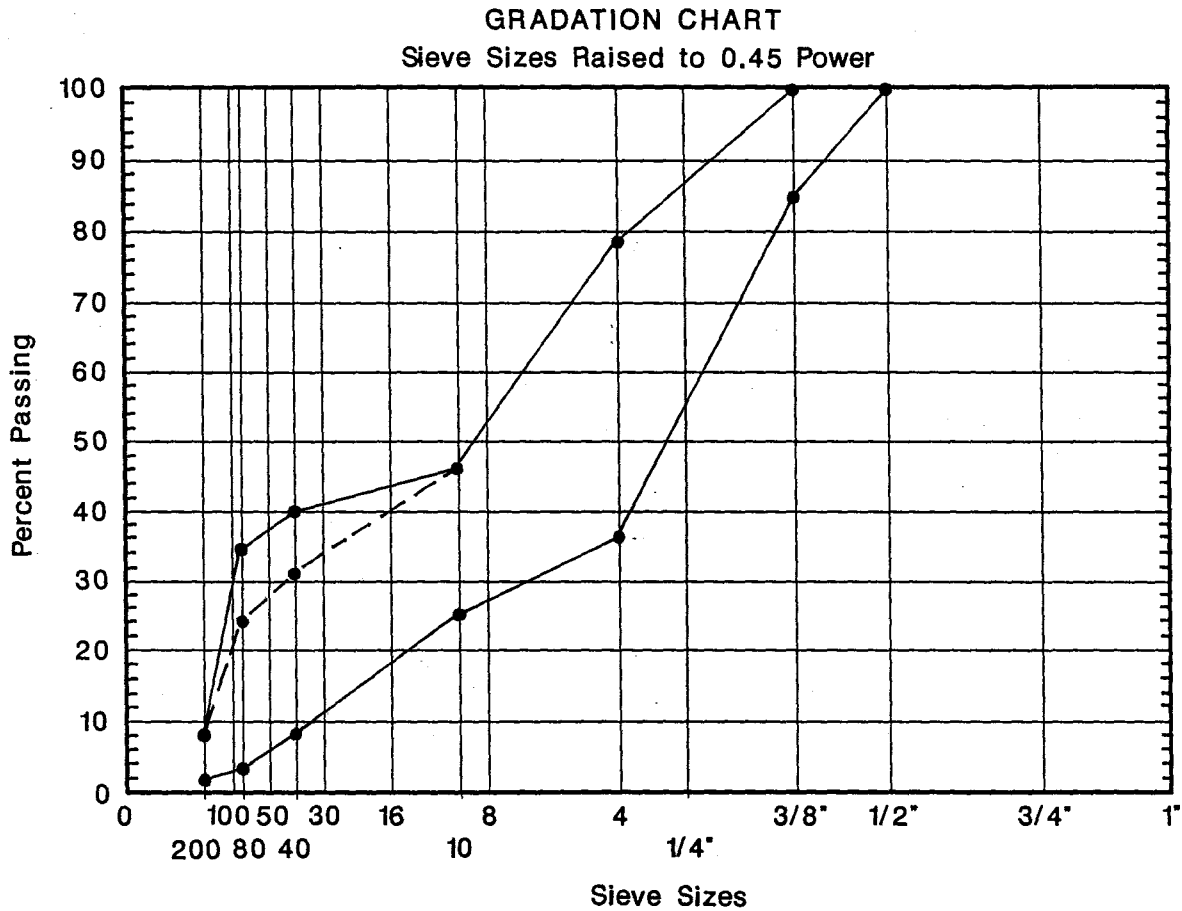


Figure 28. Aggregate Specification Limits for Item 340, Type D with Suggested Modification (dashed line) to Provide Tougher Mixtures for Construction of Intersection Approaches.

understanding of the requirements, and provide for improved overall control at the mix plant during production (50).

-- The 0.45 power gradation chart, as used in this report, is particularly useful in evaluating aggregate gradations. A straight line, plotted from the origin of the chart to the percentage point plotted for the largest sieve with material retained, represents the gradation of maximum density. Aggregate gradation should be examined on the 0.45 power chart as a routine procedure during mixture design. When a plant inspector becomes accustomed to using this chart, it may help him to recognize gradation problems early and make the necessary adjustments before large quantities of the mix are placed.

Although it is well known that siliceous gravels and sands generally produce asphalt concrete mixtures subject to plastic deformation and moisture damage, existing specifications for Item 340 do not limit the use of these natural aggregate particles. The specification requires that a minimum of 85 percent of the particles retained on the number 4 sieve have at least 2 crushed faces. This is certainly a positive move regarding the coarse aggregate, but there is no limitation placed on the fine aggregate (sand). The type and quantity of fine aggregate is critical in that it greatly influences the amount of asphalt a mixture can tolerate and the volume of air in the compacted pavement (25, 28, 31). Excessive use of natural sand is indirectly addressed in the specification by the requirement of a minimum Hveem stability. Experience, however, has shown that mixtures with satisfactory Hveem stability often will not provide satisfactory performance as surface courses on approaches to intersections carrying more than 10,000 vehicles per day. (Evidence of this was demonstrated by the 2-year routine maintenance program for intersection pavements practiced in several districts.) To provide a margin of safety, the natural aggregate particle content of mixtures to be applied at intersection approaches should not exceed about 13 percent. Quality of natural aggregate varies widely and should be considered by allowing special provisions to exceed the maximum limit when "sharp" sands with demonstrated good performance are used.

To meet gradation requirements with limited use of natural sand, it is usually required to replace these particles with "manufactured sand"

(crusher screenings with limited minus number 200). There is currently no standard specification for washed screenings, this has caused difficulties on occasion. For example, District 17 requisitioned washed screenings and the material delivered contained only 3 percent less minus number 200 material (15 instead of 18 percent) than the stone screenings usually received (which met existing Item 340 specifications). A reasonable specification for washed screenings should require near 100 percent passing the number 4 sieve and limit the amount passing the number 200 sieve to about to 6 percent. Special provisions should be considered for particular situations, depending on the character of the available screenings and sand and the other aggregates blended to produce the mix.

A target value for voids in the mineral aggregate (VMA) should be obtained through the proper distribution of aggregate gradation to provide adequate asphalt film thickness on each particle and accommodate the design air void system (48). Current Texas SDHPT specifications for Item 340 do not require a minimum VMA. Minimum allowable VMA for various nominal maximum particle sizes as recommended by FHWA (48) and McLeod (3) are shown on Figure 4. These values are based on compaction using the Marshall hammer. Optimum values of VMA using the Texas gyratory compactor need to be established. Based on findings from a recent study sponsored by the National Cooperative Highway Research Program (NCHRP) (51), it is reasonable to expect that acceptable VMA requirements using the gyratory compactor may be about 0.5 percent lower than those developed using the Marshall hammer. Krugler (52) pointed out that the greater the VMA in the dry aggregate, the greater space which is available for asphalt to coat the aggregate particles, while still leaving room for the optimum percentage of air voids. The thicker the asphalt film (up to the point where film thickness begins to interfere with stability by reducing the internal friction of aggregate interlock), the more protected the mix is from water damage and oxidative aging.

Another item that is critical to mixture performance that is not addressed in the specifications is the filler (minus number 200 aggregate) to asphalt ratio. This ratio is computed by dividing the weight percent or mass of filler by the weight percent or mass of asphalt, respectively, and

should range between a minimum of 0.6 and a maximum of 1.2. Mixtures containing preponderantly absorptive aggregates will need less filler than mixtures composed primarily of nonabsorptive aggregates. Mixtures containing high percentages of nonabsorptive gravels (even crushed) and sands will rapidly lose stability with a slight excess of asphalt. Designing at the upper limit of filler content will help reduce this sensitivity to asphalt content. Conversely, absorptive aggregates will selectively absorb the lighter, more mobile components (lower viscosity) of the asphalt more deeply into the aggregate leaving, in effect, a harder grade material to act as binder. In such cases, it would be advisable to design at the lower limit of filler content to insure adequate mixture flexibility. (When using highly absorptive aggregates, improvements in mixture quality may be gained by specifying an asphalt one grade softer than usual to provide for loss of the low viscosity materials due to absorption. Research has not been performed to establish the critical level of absorption above which one should change to a softer asphalt.)

Finally, incorporation of some or all of the above recommended changes into the Item 340 specification will result in a substantial increase in the Hveem stability. As a measure to further ensure that the mixture will withstand the special stresses applied at intersection approaches, the minimum required Hveem stability should be raised to a value between 37 and 40. A value of 37 is recommended by the Asphalt Institute (2) and the FHWA (48) for traffic volumes exceeding one million equivalent single axle loads during the design life.

METHOD OF TESTING

In the search for possible reasons for excess asphalt in paving mixtures, standard Texas test methods were investigated. Design of hot bituminous mixtures in Texas requires the use of test methods Tex-205-F (mixing) and Tex-206-F (compaction) for specimen preparation. These test methods specify a mixing temperature of 275°F and a compaction temperature of 250°F, respectively, regardless of the grade or viscosity-temperature relationship of the asphalt cement. Examination of 1988 data for AC-20

asphalts used in Texas revealed that the viscosity may range from 6 to 14 stokes at 250°F and from 2.8 to 6.8 stokes at 275°F. Based on the experience of the authors, this range of viscosities will significantly affect density of the compacted specimens. Higher viscosity will, of course, result in higher air voids. Since optimum asphalt content is selected at 97 percent density (or 3 percent air voids), it follows that the harder (at compaction temperature) asphalts will require higher asphalt contents. Now since the materials under discussion are all AC-20's, the viscosity range at high pavement service temperatures (say 140°F) is comparatively small (1610 to 2280 stokes, based on 1988 Texas asphalt data). Therefore, in service, the higher asphalt content required by the design procedure may be detrimental to resistance to plastic deformation of the mix. Furthermore, when modified asphalts are used, which often have significantly lower than usual temperature susceptibilities (or higher viscosities at the compaction temperature), the standard design procedure may require binder contents in excess of that desirable for good performance. The potential for these standard test methods to produce mixes with excess asphalt should be investigated. If it is determined that the risk is unacceptable, then the test methods should be modified to require mixing and compaction at some preselected viscosity rather than the constant temperatures. Guidelines for the Marshall design procedure (2, 53, 43) recommend a mixing temperature that provides 170 centistokes and a compaction temperature that provides 280 centistokes. Asphalt viscosity at compaction temperatures using the Texas gyratory compactor may not be as critical as viscosity when using the Marshall hammer but this supposition should be verified.

DESIGN CONSIDERATIONS

In contrast to the current empirical pavement design procedures where one is unable to determine if a paving mixture of specific strength parameters is capable of sustaining vertical and horizontal loads of varying magnitude, implementation of the mechanistic design methods presented in previous chapters provide a rational approach to design pavement of sections

capable of withstanding high tire pressures in addition to the horizontal forces produced at the surface. Furthermore, application of these methods due to their sound theoretical background warrants that detrimental shear stresses induced in the surface layer under any pavement boundary and environmental conditions will not exceed the bearing capacity of the mix.

The octahedral shear stress ratio concept can be used to evaluate the potential of an asphalt concrete overlay, over either existing asphalt concrete or portland cement concrete, to rut or deform under traffic. This ratio is based on the principle of octahedral shear stress which is a scalar or numerical representation of the stress state at any point within the pavement cross-section. This scalar quantity is calculated from the three normal and six shearing stresses acting at a given point within the pavement. Since materials fail in different modes based on the conditions of loading and temperature, and since a number of failure criteria exist, selecting the proper failure mode and criterion by which to judge failure potential is of great importance. It makes sense that the potential to deform, shove or rut in an asphalt concrete pavement at an intersection should be evaluated based on a shearing failure criterion. The authors have reviewed the literature and believe that the octahedral shear stress is the most appropriate way to evaluate the asphalt concrete deformation potential.

The procedure is summarized in the following steps:

1. Compute the maximum octahedral shear stress in the asphalt concrete overlay under the climatic, structural, and loading conditions involved. Ameri-Gaznon and Little (5) have accomplished this for the majority of conditions which will normally be encountered.

2. Measure the octahedral shearing strength of the asphalt concrete material used in the overlay at the same state of stress at which the maximum octahedral shearing stress occurs within the pavement (step 1). This can be accomplished by following the procedure for testing and analysis outlined in Appendix B.

3. Compute the ratio of the maximum octahedral shear stress developed within the asphalt concrete overlay to the octahedral shear strength of the

asphalt concrete material used in the overlay at the same stress state that occurs in the overlay at the critical point.

When the octahedral shear stress ratio (OSR) is high, the potential to deform excessively is high. When the OSR is low, the potential to deform is low. Theoretically, an OSR equal to unity represents incipient failure. However, research has shown that conditions favoring excessive deformation can result at OSR's of 0.65 in highways subjected to normal loading conditions. Although it is not possible, at present, to identify an OSR which represents a selected quantification of deformation, the OSR is an excellent way to compare various mixtures of asphalt concrete as to their relative potential to resist permanent deformation in specific structural and climatic categories.

CONSTRUCTION CONSIDERATIONS

It is possible to substantially reduce plastic deformation of the pavement by using larger nominal, maximum size aggregates, that are mixed with harder grade of asphalt (AC-30) or modified asphalt binder. Davis (40) states the largest stone size should be two-thirds the pavement thickness. Larger crushed stone mixes generally require less energy to produce and less asphalt and are, therefore, less expensive. Research (55, 56) has shown that certain polymer additives will produce a significant increase in asphalt viscosity at high pavement service temperatures while having little affect on viscosity at low pavement service temperatures. Where resistance to high shear stresses is the primary concern and low temperature cracking is of no particular concern, it may be possible to achieve positive results by incorporating a polymer additive into AC-20. The modified binder should exhibit little change in penetration at 39.2°F while exhibiting a substantial increase in viscosity at 140°F. Consequently, the use of Type C mixtures with modified asphalt in place of Type D mixtures for surface courses should substantially help to alleviate permanent deformation at intersection approaches. District 10 is evaluating the use of Type C mixes with modified asphalts at several intersections in

Tyler. Higher than usual compaction energy may be required for these mixtures.

Plant mix seals or open-graded friction courses (OGFC) are quite resistant to rutting. These mixtures may provide a viable alternative to the usual Type D dense-graded mixtures for overlays or reconstruction of intersection approaches. Additional benefits provided by plant mix seals include improved surface friction and resistance to hydroplaning and reduced glare at night. These are important factors to consider at an intersection. Mechanical properties of OGFC's can be further enhanced by use of asphalt rubber or polymer modified binders (57).

Stone filled mixtures (37), briefly described in Reference 25, should also provide excellent service on intersection approaches. Stone filled mixtures essentially consist of a small top-size, dense graded asphalt concrete mix combined with about 45 percent (by total weight of mix) of a larger single-size stone of about 3/4 inch for surface courses. A stone matrix is formed by the large stones and the voids between are filled with the fine-grained asphalt mix. The bridging effect of the large stones resist plastic deformation and further densification under traffic in a manner similar to the open-graded mixes.

PORTLAND CEMENT CONCRETE

An alternative approach to eliminate plastic flow of the pavement surface materials at intersections is the application of portland cement concrete (PCC). Rigid pavement, due to its high modulus of elasticity, will allow distribution of the applied load over a relatively larger area of the substrate, and thus result in low subgrade stresses for dowelled pavement systems. Generally, the major portion of load carrying capacity of pavements surfaced with portland cement concrete (pcc) is provided by the slab itself. This is in contrast to the flexible pavement wherein the strength of the pavement is provided by the thick layers of the subbase and/or base (1). However, design consideration must be given to the pavement jointing system, which is very important in the design process.

Jointed concrete pavements can be either plain concrete (JPC) or reinforced concrete (JRC). JPC pavements normally have dowelled transverse joints spaced 20 feet or less apart, while JRC pavements have joint spacings of greater than 20 but less than 100 feet. Slabs with a joint spacing in feet, greater than approximately twice the thickness in inches (nominally about 20 feet for highway pavements) will usually crack at intervals between 12 and 20 feet, forming a natural hinge which relieves the curling stresses (Figure 29). Reinforcing steel is normally placed at the middepth in longer slabs for which cracking is anticipated to hold these cracks tightly closed, thereby promoting load transfer across the cracks through aggregate interlock. Typically, JRC pavement designs consist of contraction joints at 40 foot intervals (Figure 30), it can be assumed that one or two transverse cracks will form between the dowelled joints. However, since curling stresses are a major factor in the thickness design, and since cracking relieves the curling stresses to the extent governed by the crack spacing, the thickness design should then be based on the length of slab between cracks rather than on the spacing between joints. Thus, all thickness designs will be based on slab lengths of 20 feet or less even though some pavements will have greater joint spacing.

One method of controlling the intermediate cracking in JRC pavements is to create a hinge at specified locations by sawing the slab (Figure 31) transversely to a depth of approximately one-fourth to one-third the slab thickness. By so doing, the hinge will form as a straight line which can be sealed in the same manner as the dowelled joints. Since it is known where the hinge(s) will form, additional reinforcing steel can be placed at these locations to aid in holding the hinge joint tightly closed and reduce the probability of rupture of the steel through corrosion. Five types of joints are available for design (shown in Figure 32), depending on the joint type and function.

For both JPC and JRC pavements a number of options can be considered to obtain economical designs. These options include:

1. shoulder type,
2. type and thickness of subbase,

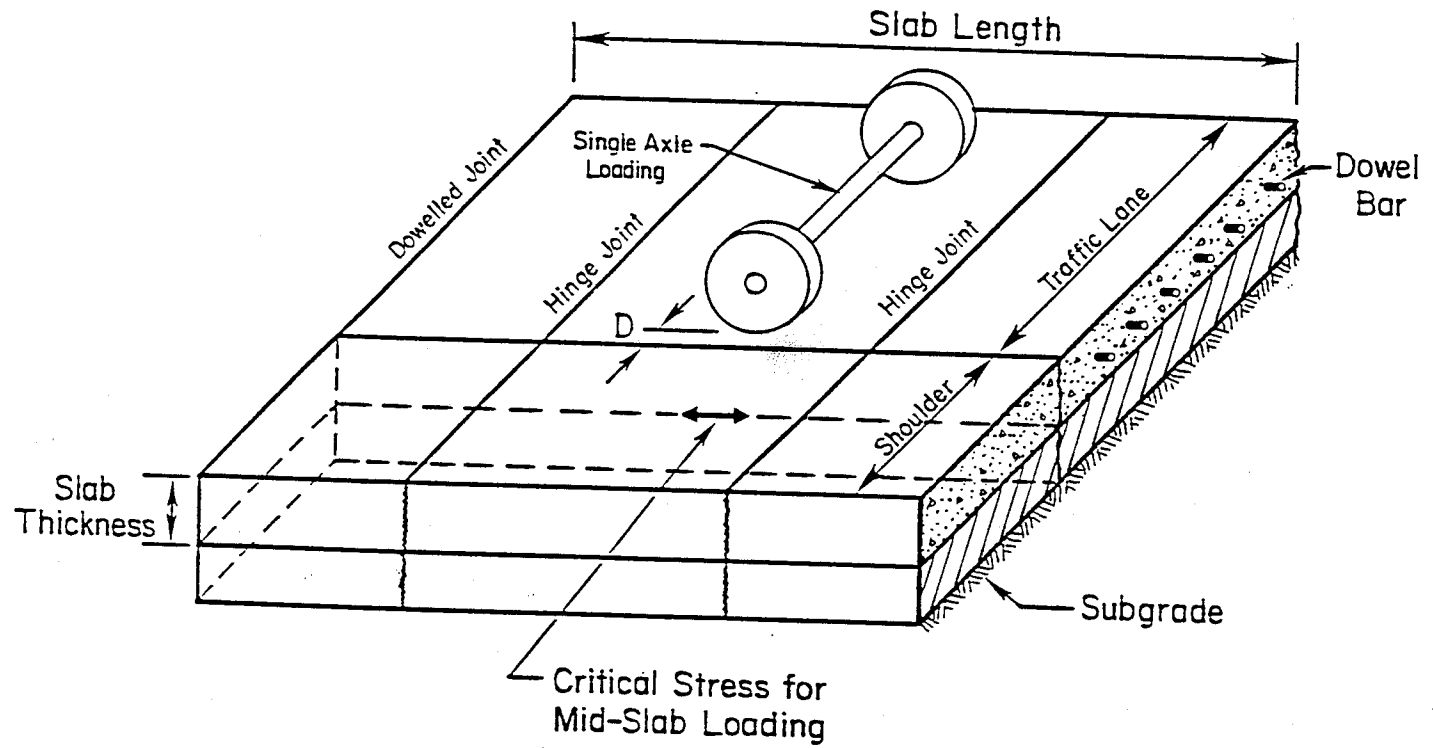
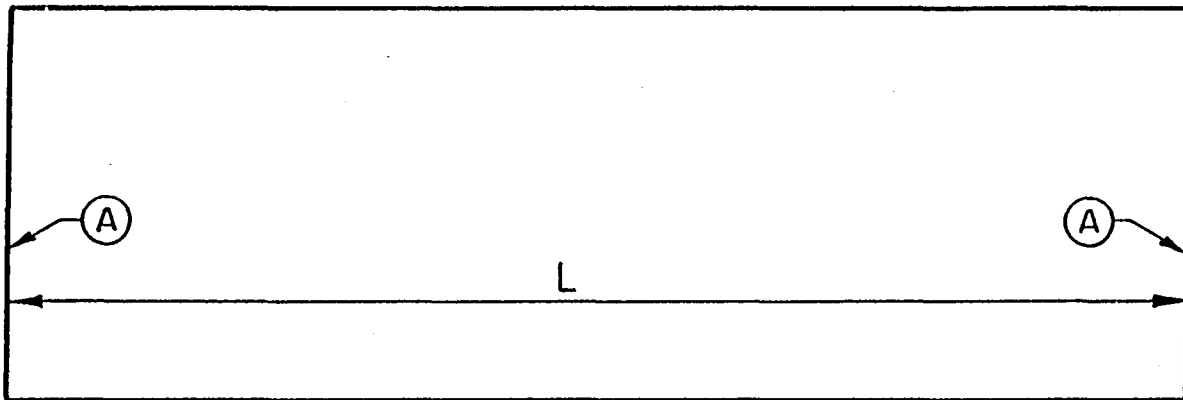
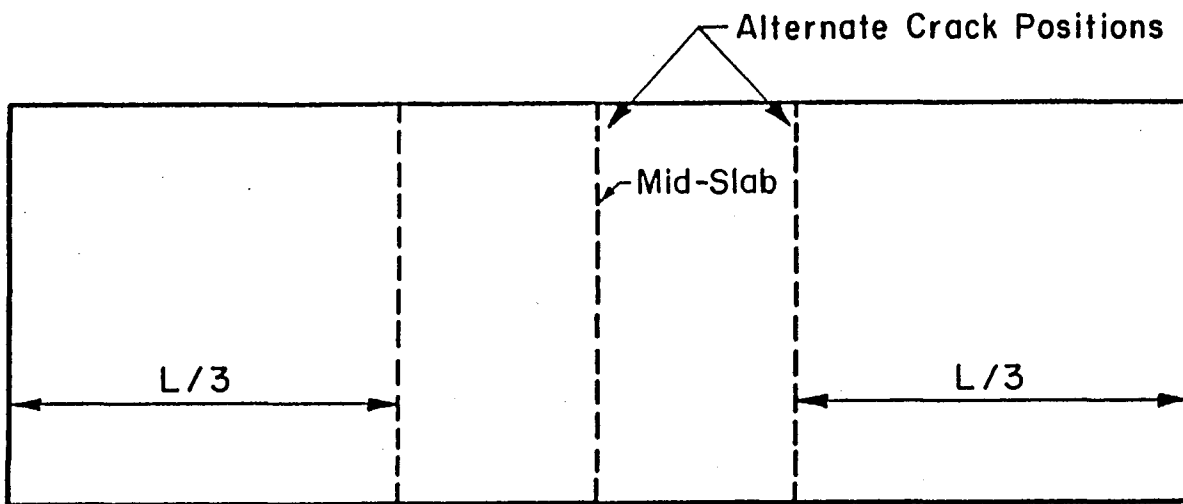


Figure 29. Slab Layout Showing Hinge Type joints to control Slab cracking.

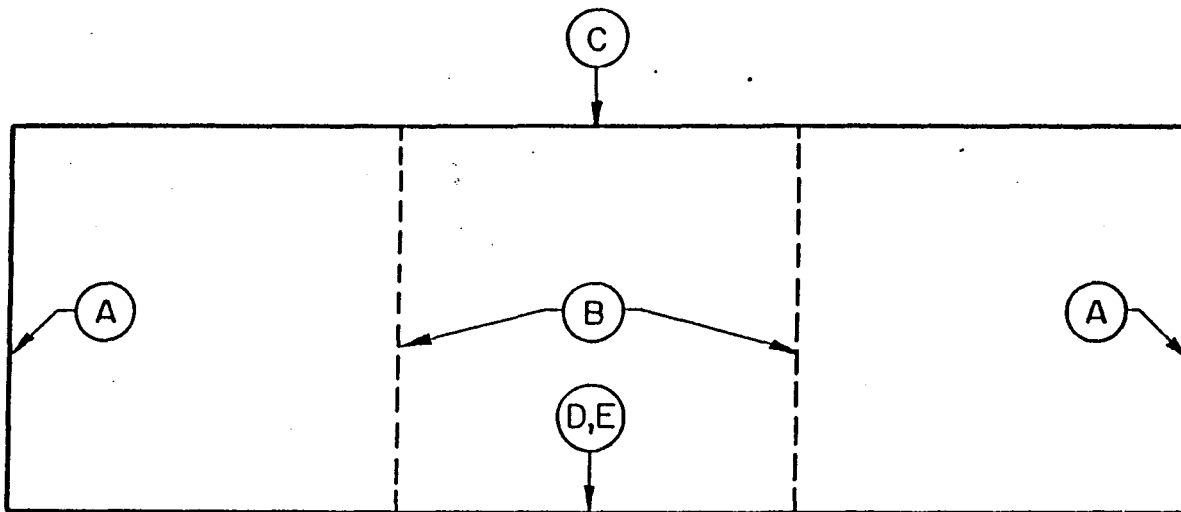


(a) 40' Jointed Slab With No Joints

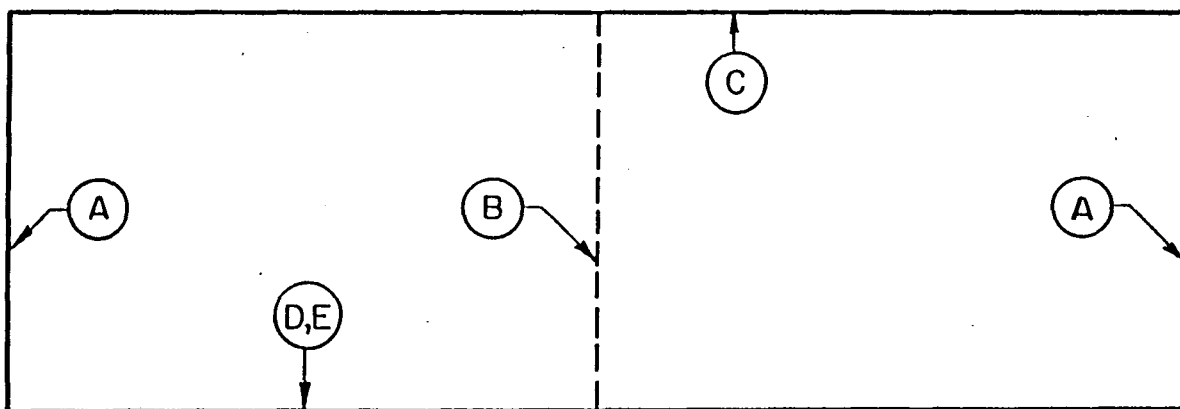


(b) 40' Joint Slab With Most Likely Crack Locations Shown as Dashed Lines.

Figure 30. Jointed Slab Layout.



(a) 40' Jointed Slab With Two Hinge Joints



(b) 40' Jointed Slab With One Hinge Joint

Figure 31. Joint Spacing Layout with Hinge Joints.

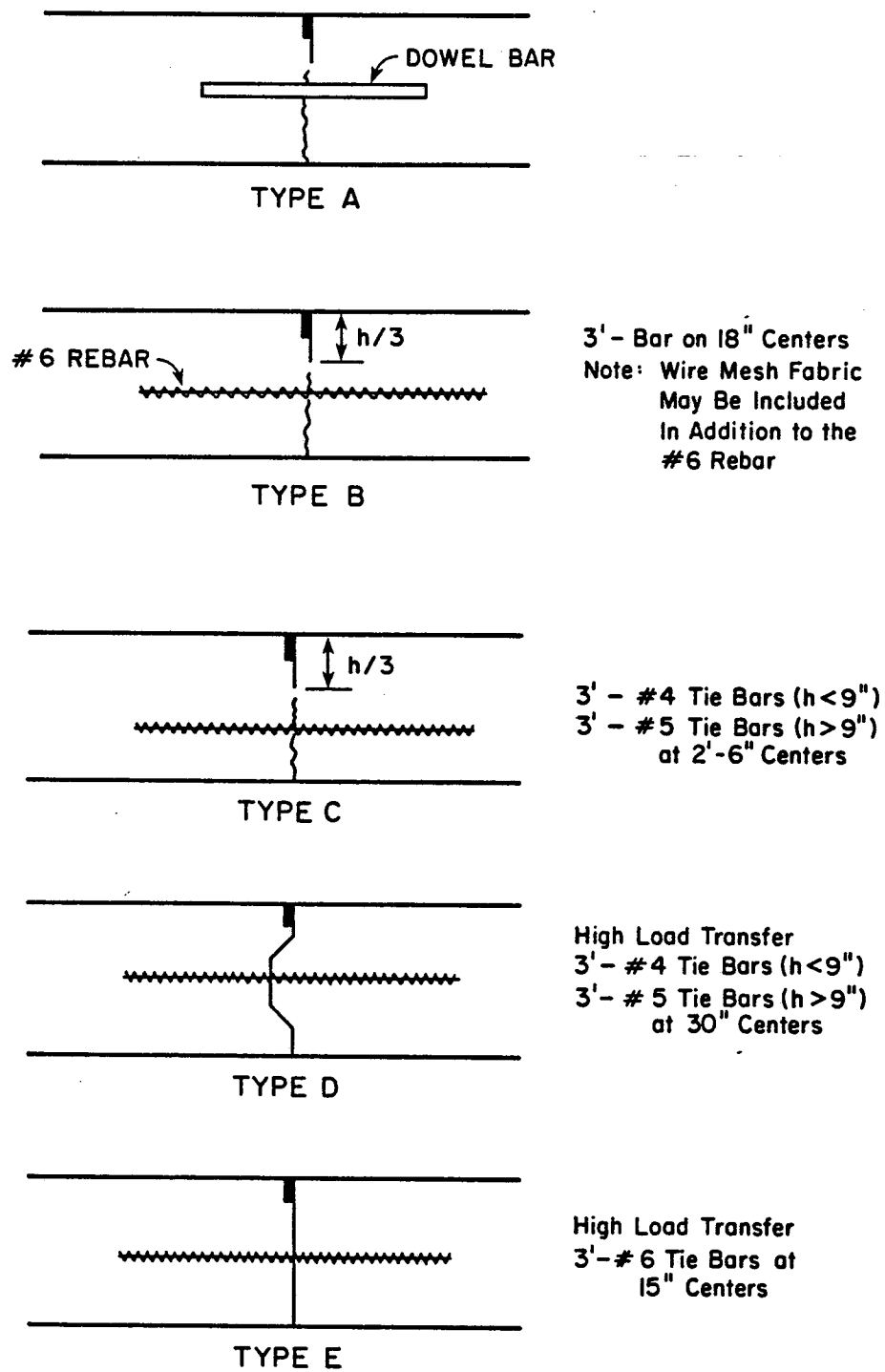


Figure 32. Joint types used in Rigid Pavement Construction.

3. strength of the concrete,
4. surface and subsurface drainage,
5. design life and reliability of design, and
6. joint spacing and type of load transfer.

To achieve an effective design, all of these options must be considered simultaneously as a part of a total pavement system. To consider these options one at a time outside the total pavement system would result in an uneconomical and inefficient design.

The one approach to design which will permit the simultaneous consideration of these multiple options in a rational manner is a mechanistic based approach. All other approaches to design bring the options into the process in an arbitrary and empirical manner, which may or may not consider the interactions between option variables. Mechanistic based procedures permit the designer to weigh the effects of one variable against the effects of other variables in a rational manner when considering the effectiveness of a design option. By weighing one variable against others, it is possible to determine the cost-effectiveness of each design option and thus develop an optimum system design.

The design should be based on the elimination of transverse cracking of the slabs (the predominant mode of failure of the PCC slabs in the AASHO Road Test) due to fatigue of the concrete caused to repeated applications of load and curling stresses. Pavement performance is evaluated by the percent slabs which have cracked, with a limiting slab cracking depending on the level of cracking desired. Correlation between the incidence of transverse cracking and the calculated fatigue damage have been developed from pavements in service.

Although plastic deformation is not a problem in specially stressed sections constructed with PCC materials, care must be exercised to provide adequate and drain-free base and subbase materials to avoid development of pumping which can create a void space under the slab. The functions of the subbase for rigid pavement performance are outlined in Table 13.

Table 13. Functions of Rigid Pavement Subbases

-
1. To provide a stable construction platform
 2. To control the depth of frost penetration
 3. Prevent erosion of the pavement support
 4. Provide uniform slab support
 5. Facilitate drainage
 6. Provide increased slab support
-

In summary, the structural weakness in PCC pavements is at or near the joints. Rigid pavement failure due to overloading will not take the form of rutting or shoving but typically may be manifested as a joint-related failure. Therefore, it is important to develop a balanced design between the interior slab and edge slab positions such that cracking due to pumping, curling, and other load induced causes leading to excessive tensile stresses in the slab will be controlled within tolerable levels.

CONSTRUCTION SEQUENCE

It is generally desirable to construct an entire bituminous pavement project with one selected mixture in order to produce a uniform product. With this approach, certain segments of some pavements, such as intersections, that are subjected to high concentrations of vehicle maneuvering are underdesigned.

An efficient and possibly cost-effective approach to alleviate permanent deformation at critically stressed pavement sections, such as intersections, is to employ a sequential construction technique. In this approach, the intersections and other critical areas which receive a higher concentration of vehicle maneuvering are constructed first with a preselected mixture that is designed to conform with the intensity of the traffic and applied loads. Once the stage construction of these areas is completed, construction of the tangent sections may begin using the normal mixture that is compatible with the type of traffic to which it is exposed.

This procedure will be particularly valuable for arterials where major intersections are separated by a considerable distance.

On occasion, it may be advantageous to let bids separately for construction or overlaying of intersections at several widely distributed locations. One would need to insure that one contractor did not interfere with subsequent work on tangent sections that may be performed by another contractor.

GEOMETRICS

Traffic monitoring was conducted at several intersections to estimate the distance from the intersection where braking force is first applied to reduce vehicle speed and then further applied to bring a vehicle to a complete stop. As mentioned previously, rut depths were measured at various pavement intersection approaches. This information was used to estimate the average length of the damaged zone of typical intersection approaches and thus to estimate the length of approach that should receive specially designed pavements. Evidence indicates that the typical length of intersection approach that should receive special treatment will range from 100 to 250 feet. The length will depend upon level and speed of traffic, traffic control methods, and, of course, the average length of the queue line that forms during stoppages.

ECONOMIC CONSIDERATIONS

The potential for significant economic benefits appears promising when intersection approaches are designed and constructed specifically to accommodate the special stresses to which they are subjected. The alternative has often been to maintain intersection approaches with overlays or level-up courses every two years. Cost comparisons of these alternatives on both a first-cost and life-cycle basis are of interest to the engineer and should be considered when selecting the optimum rehabilitation alternative for a particular intersection approach. A simple example to illustrate the potential savings is given below.

Based on the findings in this study, it seems reasonable to assume that an improperly designed intersection will need to be maintained by overlaying or milling or both every two years. As a basis for comparison, assume an intersection approach consisting of four 12-foot lanes 150 feet in length will be (1) overlaid with 1 inch of asphalt maintenance mix every 2 years or (2) designed and built with special hot mix to serve without maintenance for 10 years. Table 14 gives approximate costs of the materials, equipment, and labor for the two alternatives. It can be seen from this oversimplified example that a savings of \$4920 per intersection approach can be realized every 10 years when an approach pavement is built during initial construction or rehabilitation to withstand the special stresses applied.

Additional benefits that can be gained by considering the special stresses associated with intersections during pavement design and construction include: (1) improved driver safety due to good condition of pavement surface (no rutting or flushing and adequate surface friction in wet weather) and (2) no buildup of maintenance mix which is often of lower quality than hot mix. When pavement user cost is considered at an intersection the cost doubles since traffic flow on at least two different thoroughfares is interrupted.

Table 14. Pavement Treatment Alternatives and Cost Comparisons.

Basis for Comparison:

Four traffic lanes 12 feet wide and 150 feet long

Maintenance Alternative: One-inch thick level-up course of asphalt mix placed by maintenance forces every 2 years for 10 years. Assume 1 day required to perform maintenance each time.

Materials - 42 tons HMCL at \$20/ton	\$ 850
Equipment - 2 dump trucks at \$30 ea/day	60
1 sign truck at \$30/day	30
1 steel wheel roller at \$20/day	20
1 distributor truck at \$30/day	30
1 grader at \$50/day	50
	<hr/>
Total Equipment	\$ 190
Labor - 1 crew leader at \$100/day	100
2 maint. operators at \$80 ea/day	160
3 maint. workers at \$65 ea/day	195
1 flagman at \$50/day	50
	<hr/>
Total Labor	\$ 505
TOTAL DAILY COST	\$ 1,545

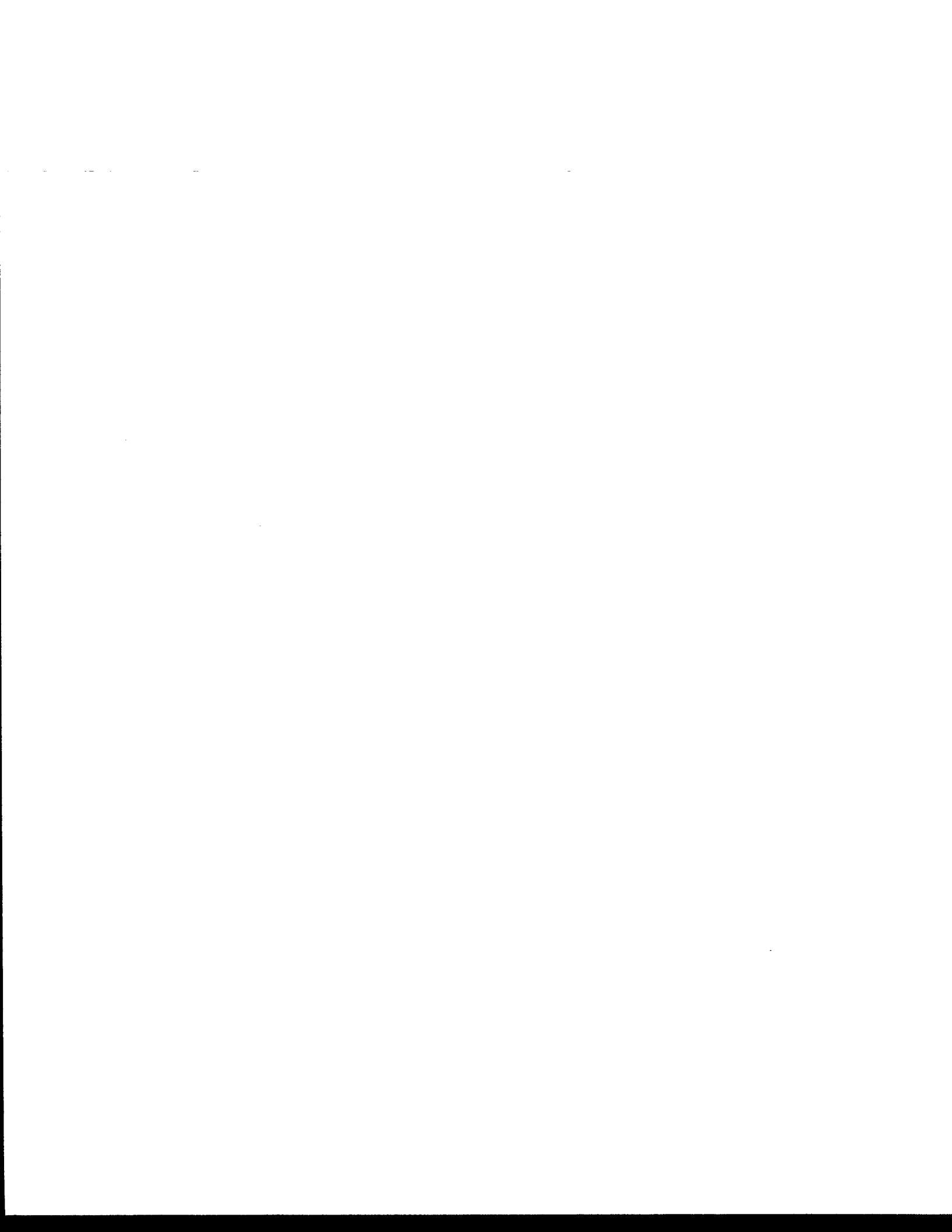
Assume 4 repetitions of the above maintenance activity will be performed in 10 years.

TOTAL 10 YEAR COST	\$ 6,180
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Ten-Year Design Alternative: During construction, apply 3 inches of special hot mixed asphalt concrete (HMAC) designed to perform satisfactorily without maintenance for 10 years.

Materials - Additional Cost of 126 tons of Special HMAC, \$10/ton	\$ 1,260
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Savings = \$ 6,180 - \$ 1,260 = \$ 4,920 per intersection per 10 years



CONCLUSIONS AND RECOMMENDATIONS

Based on findings from a review of current literature as well as field and laboratory investigations, the following conclusions and recommendations are tendered.

CONCLUSIONS

1. The most common form of distress associated with failure of asphalt concrete intersection approach pavements was plastic deformation manifested in the form of rutting with shoving and flushing in some cases. In all cases investigated, rut depths more than 250 feet from the intersection were practically negligible.
2. The leading materials related cause of intersection pavement failure was binder content in the asphalt concrete in excess of that required by the optimum mixture design. Some variation in asphalt content in a mix plant appears inevitable. It also appears that on occasion asphalt content is arbitrarily increased by the engineer to facilitate compaction.
3. Most of the mixtures studied contained relatively high percentages of natural (uncrushed) sand. The smooth, rounded, nonporous, glassy character of these fine aggregates cause the mixture to be sensitive to asphalt content and weak in shear strength which thus imparts a higher propensity for permanent deformation. Approximately 30 percent minus number 40 sieve size material, which was largely natural sand, was found in all the problem intersections. (State specifications for Item 340, Type D allow up to 40 percent passing the number 40 sieve.) Gap-graded mixtures containing rounded particles at the no. 40 sieve tend to be tender.
4. Aggregate gradations appeared to be very dense for some intersection pavements that experienced early failure. Dense aggregate gradations leave little room for asphalt binder, and the mixture may become unstable with a slight excess of asphalt. This is particularly true for fine-grained asphalt mixtures.

5. Air void contents obtained from almost all the rutted intersection pavements were comparatively low (less than 3 percent), particularly in the wheelpaths. This indicates that either the mixture designs were too dense or that they were overcompacted during construction such that additional densification by traffic caused the mixtures to become unstable soon after construction and exhibit plastic flow (rutting).
6. The filler (minus #200) content of the paving mixtures was generally low (<4%). This condition enhances sensitivity to binder content. An increase in the amount of filler will stiffen the binder and thus increase the bulk viscosity of the mixture, which may help in diminishing permanent deformation potential. Caution should be used however, because as filler content is increased, mixture flexibility (resistance to cracking) may be diminished. Filler to asphalt ratio should range between 0.6 and 1.2.
7. Many districts had established a routine two-year maintenance program, wherein most intersection approaches in the district with significant traffic received treatment every other year. This is an indicator of the severity of the problem with intersection pavement service life.
8. The potential for significant economic benefits appears promising when intersection approaches are designed and constructed specifically to accommodate the special stresses to which they are subjected. If a segment of pavement to be built or rehabilitated contains a large number of intersections such that it is not economically feasible to apply special pavements at the intersections, then the complete project should be designed and built to withstand the most damaging loads.

RECOMMENDATIONS

1. Reduce the quantity of sand size (minus #10, plus #200) particles allowed in Item 340 mixtures to be used on intersection approach pavements. Generalized examples of these recommended changes are

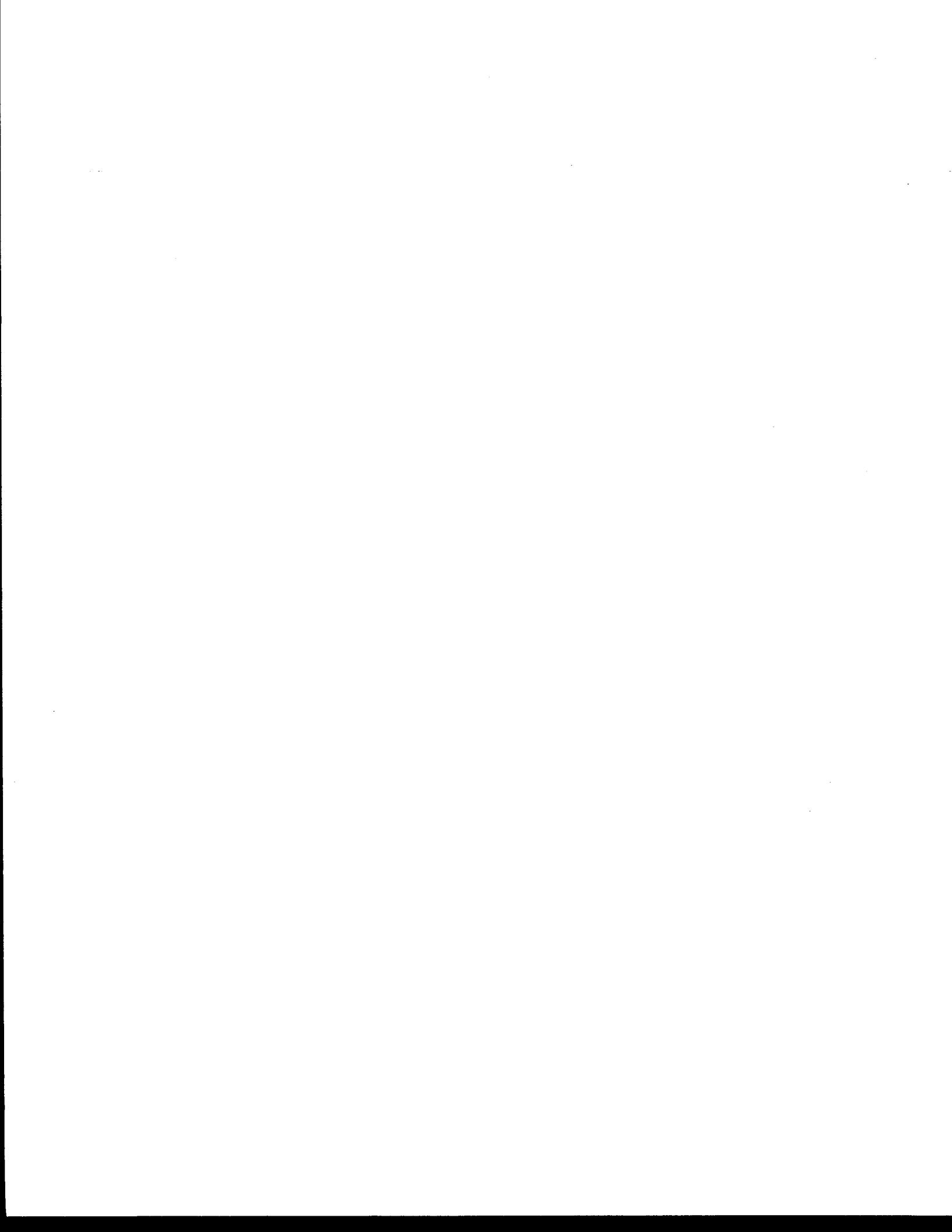
- shown for Type C and D mixtures in Figures 27 and 28. Additional research will be necessary to precisely determine optimum gradations.
2. Limit the natural (uncrushed) sand content of Item 340, Type D mixes to be used on the surface of intersection pavements to about 13 percent. Special provisions should be allowed for "sharp" sands that have demonstrated good performance wherein they may exceed the specified value.
 3. Convert aggregate gradation specifications in Item 340 (Texas SDHPT specifications) from passing-retained to total percent passing. This should provide for a better general understanding of the specification and enhance mixture design and construction controls.
 4. Train design and construction quality control personnel on the use of the 0.45 power gradation chart. This could help recognize gradation problems early and indicate where adjustments are necessary.
 5. For Item 340, institute a specification for voids in the mineral aggregate (VMA) considering the fact that the gyratory compactor generates a specimen that simulates final density after significant traffic. Optimum VMA values for gyratory compacted specimens may be slightly lower than those proposed by FHWA and the Asphalt Institute. All pertinent factors should be carefully examined before a VMA specification is prepared.
 6. In a special specification, require a minimum Hveem stability of 40 for mixes to be applied on high traffic volume intersection approaches. This is an indirect method of assuring good aggregate quality.
 7. The FHWA recommends a filler to asphalt ratio of 0.6 to 1.2 for asphalt concrete mixtures (Item 340). Filler to asphalt ratio should be examined routinely during asphalt mix design and construction.
 8. Use of comparatively large maximum size aggregate with asphalt modifiers to increase viscosity at higher pavement service temperatures may offer cost-effective alternatives to prolong intersection pavement life. Options include dense-graded large stone mixes (Item 340, Type C or B), stone filled mixes and plant mix

seals. The National Asphalt Pavement Association recommends a maximum aggregate size of 3 inches or up to two-thirds the pavement layer thickness, whichever is smaller.

9. Establish a specification for "washed" stone screenings which would require near 100 percent passing the #4 sieve and limit the minus #200 material to a maximum of about 6 percent. The absence of this specification has caused problems in procuring materials of adequate quality to replace natural (uncrushed) sands. Development of this specification was not within the scope of this study.
10. Use a rational approach for mixture design to increase the probability of producing a mix that will give satisfactory service. A rational approach for mixture design using octahedral shear strength ratio is suggested herein and sample calculations are provided.
11. Consider specifying constant asphalt viscosities during mixing and compaction rather than constant temperatures for standard test methods Tex-205-F and Tex-206-F. Use of the mixing temperature of 275°F and the compaction temperature of 250°F for hard or modified asphalts with the standard Texas mix design procedure may result in excess binder content which could lead to rutting or flushing. The Asphalt Institute recommends mixing and compaction temperatures that provide 170 and 280 centistokes, respectively. A detailed investigation of the effects of temperature on gyratory compaction should be performed.
12. Consider the use of portland cement concrete for intersection approaches where economic analyses of the alternatives indicate it is the appropriate material.
13. The length of an intersection approach that should receive special treatment may range from 100 to 250 feet, depending upon the traffic speed and density and length of the typical queue line that develops during stoppages.
14. Employ a sequential construction technique where all intersection approaches within the project are completed prior to the remainder

of the job using a special, tough mix to accommodate the special stresses.

15. Utility of the changes in specifications, test methods and construction techniques recommended herein needs to be verified through a series of controlled field and laboratory experiments. The 18-month study described herein has merely estimated the magnitude of the problem of early intersection failure, identified some of the primary causes of early failure and recommended some changes in materials specifications, mixture design methods and construction quality control measures.



REFERENCES

1. American Association of State Highway and Transportation Officials, AASHTO Guide for Design of Pavement Structures, Washington, D.C., 1986.
2. The Asphalt Institute, "Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types," Manual Series No. 2, May, 1984.
3. McLeod, N.W., "Designing Standard Asphalt Paving Mixtures for Greater Durability," Proceedings, Canadian Technical Asphalt Association, Volume 16, 1971.
4. Lottman, R.P., "Laboratory Test Method for Predicting Moisture-Induced Damage to Asphalt Concrete," Transportation Research Record 843, Transportation Research Board, Washington, D.C., 1982.
5. Ameri-Gaznon, M., and D.N. Little, "Permanent Deformation Potential in Asphalt Concrete Overlays Over Portland Cement Concrete Pavements," FHWA Report No FHWA/TX-88/452-3F, Austin, Texas, November, 1988.
6. Carpenter, S.H., and T.J. Free, "Characterizing Permanent Deformation in Asphalt Concrete Overlay Over Portland Cement Concrete Pavements," Transportation Research Record 1070, Washington, D.C., 1986.
7. McLeod, N.W., "Rational Design of Bituminous Paving Mixtures with Curved Mohr Envelopes," Proceeding of Association of Asphalt Technologists, Vol. 21, January 28-30, 1952.
8. Terzaghi, K. and R.B. Peck, Soil Mechanics in Engineering Practice, Second Edition, John Wiley and Sons, New York, 1968.
9. Nijboer, L.W., "Plasticity as a Factor in The Design of Dense Bituminous Road Carpets," Elsevier Publishing Company, Inc., New York, Amsterdam, London, Brussels, 1948.
10. Goetz, W.H., and C.C. Chen, "Vacuum Triaxial Technique Applied to Bituminous Aggregate Mixtures," Proceedings of Association of Asphalt Technologists, Vol. 19, February, 1950.
11. Middleton, D.R., F.L. Roberts, and T.C. Chavala, "Measurement and Analysis of Truck Tire Pressures on Texas Highways," Transportation Research Record 1070, Washington, D.C., 1986, pp. 1-8.
12. Hector, M.C., "A Possible New Approach to Develop a Mohr Diagram," Proceedings of Association of Asphalt Technologists, Vol. 22, January 26, 27 and 28, 1953.

REFERENCES (continued)

13. Carpenter, S.H., "New Bituminous Mixture Analysis for Overlay," Presented at FHWA Training Course, Denver, Colorado, March 13, 1986.
14. Livneh, M., and E. Shklarsky, "The Bearing Capacity of Asphalt Concrete Carpets Surfacing," Third International Conference - Structural Design of Asphalt Pavements, University of Michigan, Vol. 1, 1972.
15. Ullidtz, P., and Elsevier, "Pavement Analysis," Institute of New York, 1987.
16. Molenaar, A.A.A., "Structural Performance and Design of Flexible Road Construction and Asphalt Concrete Overlays," aan de Technische Hogeschool Delft, Netherlands, 1983.
17. Tielking, J.T., and F.L. Roberts, "Tire Contact Pressure and its Effect on Pavement Strain," Journal of Transportation Engineering, Vol. 113, January, 1987.
18. Harrison, W.J., J.J. Wardle, and C.M. Gerrard, "Computer Programs for Circle and Strip Loads on Layered, Anisotropic Media," Division of Applied Geomechanics, Commonwealth Scientific and Industrial Research Organization, Australia, 1972.
19. McLeod, N.W., "The Rational Design of Bituminous Paving Mixtures," Proceedings, Highway Research Board, Vol. 29, 1949.
20. Brown, E.R., "Experiences of Corps of Engineers in Compaction of Hot Asphalt Mixtures," Placement and Compaction of Asphalt, ASTM, STP829, American Society of Testing and Materials, 1984.
21. Uzan, J., M. Livneh, and Y. Eshed, Investigation of Adhesion Properties Between Asphalt Concrete Layers, Proceedings of Association of Asphalt paving Technologists, Vol. 47, 1978, pp. 495-521.
22. Sargios, M., "Pavements and Surfacing for Highways and Airports," John Wiley and Sons, New York, Toronto, 1975.
23. Texas State Department of Highways and Public Transportation, "Standard Specifications for Construction of Highways, Streets and Bridges," Adopted September 1, 1982, pp. 239-247.
24. Herrin, M., and W.H. Goetz, "Effect of Aggregate Shape on Stability of Bituminous Mixes," Proceedings, 33rd Annual Meeting of Highway Research Board, January, 1954.

REFERENCES (continued)

25. Button, J. W., and D. Perdomo, "Investigation of Rutting in Asphalt Concrete Pavements," Interim Report FHWA/YX89 1121-1, Texas Transportation Institute, College Station, Texas, March 1989.
26. Kandhal, P.S., "Evaluation of Baghouse Fines in Bituminous Paving Mixtures," Association of Asphalt Paving Technologists, Vol. 50, San Diego, California, February, 1981.
27. Monismith, C.L., "Influence of Shape, Size, and Surface Texture on The Stiffness and Fatigue Response of Asphalt Mixtures," Special Report 109, Highway Research Board, Washington, D.C., 1970.
28. Foster, C.R., "Dominant Effect of Fine Aggregate on Strength of Dense-Graded Asphalt Mixes," Special Report 109, Highway Research Board, Washington, D.C., 1970.
29. Benson, F.J., "Effects of Aggregate Size, Shape, and Surface Texture on the Properties of Bituminous Mixtures," Special Report 109, Highway Research Board, Washington D.C., 1970.
30. Herrin, M. and W.H. Goetz, "Effect of Aggregate Shape on Stability of Bituminous Mixes," Proceedings, Highway Research Board, Volume 33, 1954, pp. 293-308.
31. Griffith, J.M. and B.F. Kallas, "Influence of Fine Aggregates on Asphaltic Concrete Paving Mixtures," Proceedings, Highway Research Board, Volume 37, 1958, pp. 219-255.
32. Lottman, R.R. and W.H. Goetz, "Effect of Crushed Gravel Fine Aggregate on the Strength of Asphaltic Concrete Surfacing Mixtures," National Sand and Gravel Association, Washington, D.C., Circular No. 63, 1956.
33. Shklarsky, E. and M. Livneh, "The Use of Gravels for Bituminous Paving Mixtures," Proceedings, Association of Asphalt Paving Technologists, Volume 33, 1964, pp. 584-610.
34. Wedding, P.A. and R.D. Gaynor, "The Effects of Using Gravel as the Coarse and Fine Aggregate in Dense-Graded Bituminous Mixtures," Proceedings, Association of Asphalt Paving Technologists, Volume 30, 1961, pp. 469-488.
35. Lefebvre, J., "Recent Investigations of Design of Asphalt Paving Mixtures," Proceedings, Association Asphalt Paving Technologists, 1957, pp. 297-309.

REFERENCES (continued)

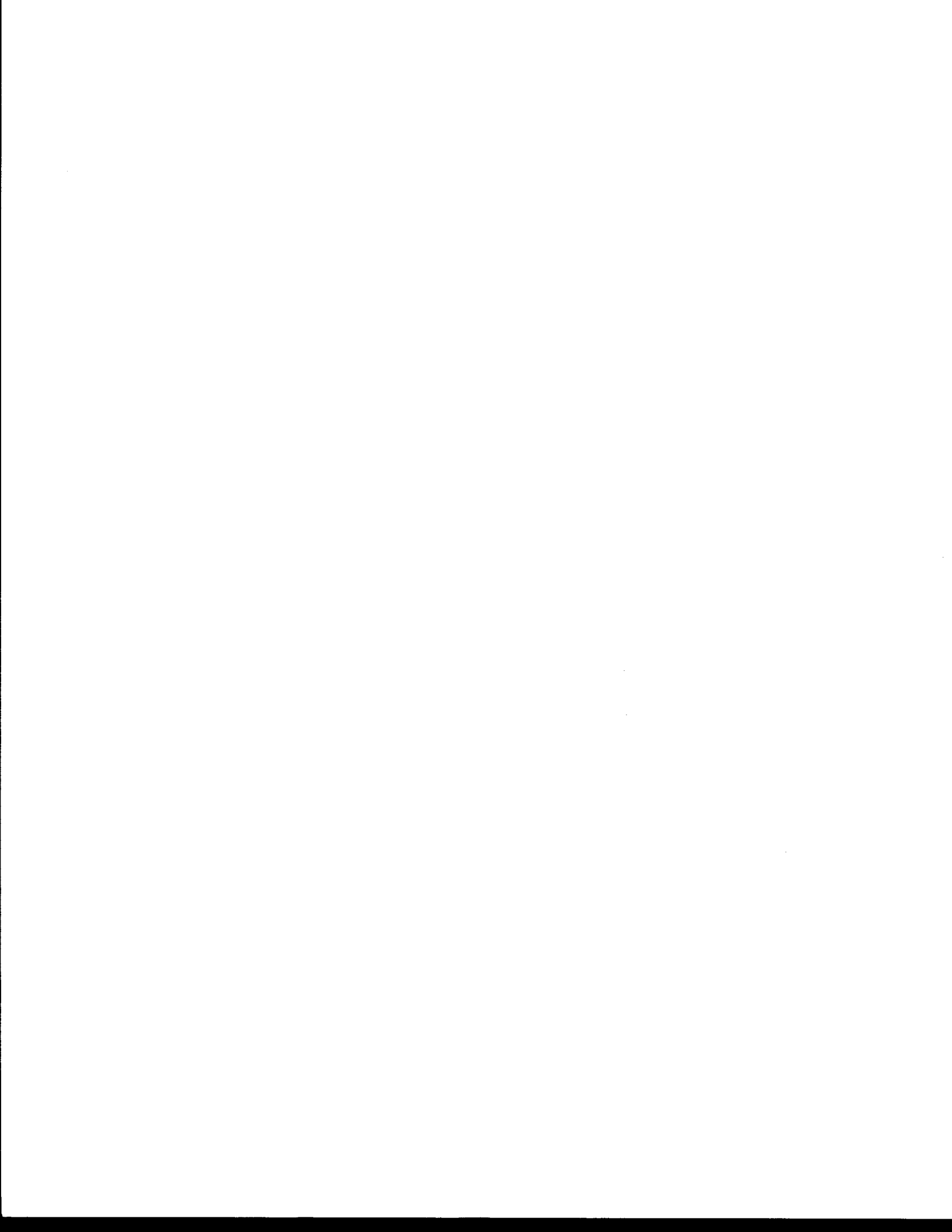
36. Kandhal, P.S., "Investigation of Pavement Rutting and Shoving Near Intersections In Pittsburgh Area," Commonwealth of Pennsylvania, Department of Transportation Bureau of Contract Quality Control, Materials and Testing Division, March 1982.
37. Acott, M., "Today's Traffic Calls for Heavy Duty Asphalt Mixes," Roads and Bridges, Volume 26, No. 1, January 1988, pp. 39-45.
38. Smith, R.D., R.P. Humer and A.B. Webb, State DOT's Choose Large Aggregate Asphalt Mixes for Stability and Durability," Asphalt Magazine, The Asphalt Institute, Volume 2, Fall, 1989.
39. Asphalt Institute, Technical Bulletin No. 5, Research Park Drive, Lexington, Kentucky, 1989.
40. Davis, R.L., "Large Stone Mixes: A Historical Insight," IS103/88, National Asphalt Pavement Association, Riverdale, Maryland, 1989.
41. Acott, M., "The Design of Hot Mix Asphalt for Heavy Duty Pavements," Quality Improvement Series 111/86, National Asphalt Pavement Association, Riverdale, Maryland, 1988.
42. Acott, M., "Large Stone Mixes - Making a Comeback Across the Country," Hot Mix Asphalt Technology, National Asphalt Pavement Association, Riverdale, Maryland, Summer 1989.
43. Crans, J., I. Ishai, and A. Sides, "Durability of Bituminous Paving Mixtures as Related to Filler Type and Properties," Proceedings, Association Asphalt Paving Technologists, Vol. 50, San Diego, California, February, 1981.
44. Kalcheff, I.V., and D.G. Tunicliff, "Effects of Crushed Stone Aggregate Size and Shape on Properties of Asphalt Concrete," Proceedings, Association Asphalt Paving Technologists, Vol. 51, Kansas City, Missouri, February, 1982.
45. Cople, F., "Premature Rutting Can Be Prevented," Material and Engineering Technology and Engineering Science No. 29, Materials and Technology Division, Michigan Department of Transportation, March 1989.
46. Huber, G.A., "Deformation? We've Already Got the Solution," Asphalt Magazine, The Asphalt Institute, Lexington, Kentucky, Volume 2, Fall, 1989
47. Huber, G.A., "Saskatchewan Highways and Transportation Improves Asphalt Pavement Performance," Asphalt Magazine, The Asphalt Institute, Lexington, Kentucky, Spring 1989.

REFERENCES (continued)

48. FHWA Technical Advisory T5040.27, "Asphalt Concrete Mix Design and Field Control," Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., March 1988.
49. Chastain, W.E., and J.E. Burke, "State Practices in the Use of Bituminous Concrete," HRB Bulletin No. 160, Highway Research Board, Washington, D.C., 1957, pp. 1-107.
50. Harm, E.E., "Illinois Rutting Report," Paper prepared for presentation at the 13th Annual Asphalt Paving Seminar, Ft. Collins, Colorado, December 1985.
51. Von Quintus, H.L., J.A. Scherocman, C.S. Hughes, and T.W. Kennedy, "Development of Asphalt-Aggregate Mixture Analysis System: AAMAS," National Cooperative Highway Research Program, Washington, D.C., September 1988.
52. Krugler, P.E., "Voids in the Mineral Aggregate," Technical Quarterly, Volume 5, Issue 1, Texas State Department of Highways and Public Transportation, Austin, Texas, June 1989.
53. AASHTO Designation: T245-82, "Resistance to Plastic Flow of Bituminous Mixtures Using the Marshall Apparatus," Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, 14th Edition, August 1988.
54. ASTM Designation: D1559-82, Resistance to Plastic Flow of Bituminous Materials Using the Marshall Apparatus, Annual Book of Standards, Volume 04.03, Road and Paving Materials; Traveled Surface Characteristics, 1989.
55. Button, J.W. and D.N. Little, "Asphalt Additives for Increased Pavement Flexibility," Report No. FHWA/TX-87/471-2F, Texas Transportation Institute, Texas A&M University, November 1987.
56. Little, D.N., J.W. Button, R.M. White, E.K. Ensley, Y. Kim, and S.J. Ahmed, "Investigation of Asphalt Additives," Report No. FHWA/RD-87/001, Texas Transportation Institute, Texas A&M University, November 1986.
57. Sainton, A., "Advantages of Asphalt Rubber Binder for Porous Asphalt Concrete," Beugnet Group, France, Paper submitted for presentation at 69th Annual Transportation Research Board, January 1990.



APPENDIX A



QUESTIONNAIRE

The Texas Transportation Institute (TTI) at Texas A&M University is involved in a research project on specially stressed pavements for the State Department of Highways and Public Transportation. The overall purpose of the study is to develop techniques that can be employed in a cost-effective manner to design and build intersections that will perform equally as well as the tangent sections. Among other benefits anticipated from the study, one can find the following:

- Reduced pavement distress (corrugations and rutting) at intersections.
- Extended service life of intersecting pavements.
- Increased driver safety
- Improved economics resulting from decreased maintenance activities at intersections.

A work plan has been outlined and is in effect. The reason for contacting you is directly related to one of the major tasks contained in the work plan. In order to supply realistic input data for the computer models to be used in the analysis, a selection of field sites with successful and unsuccessful intersections needs to be investigated.

Briefly describe any pavement distress-related intersection problems (rutting and/or corrugation) which you consider are of major importance which are present in an HMAC layer not older than 2 years. We are primarily interested in distress occurring within the HMAC layer. If you are experiencing such problems, please select no more than three such intersections and supply the following information:

1. Location of intersection
2. Traffic volume and type (high, medium or low; approximate percent of trucks) (if necessary, we will get details from D-10)
3. Pavement materials (asphalt, aggregate, base)
4. Subgrade conditions (soil type, unique features)
5. Location of major problems within intersection
6. Types of problems (rutting, corrugation, others...)
7. Possible causes
8. Sketch of intersection including traffic control
9. Approach speed

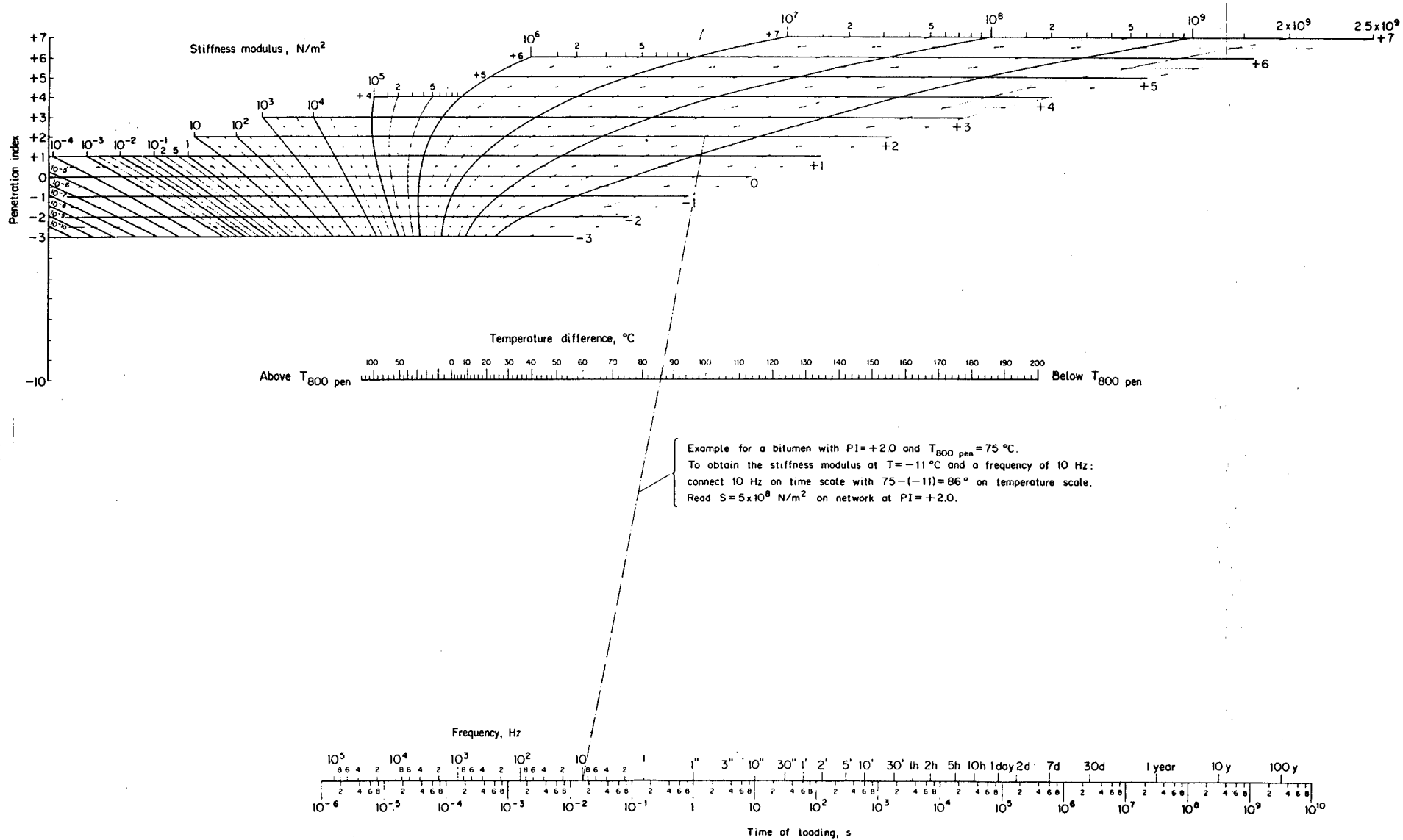
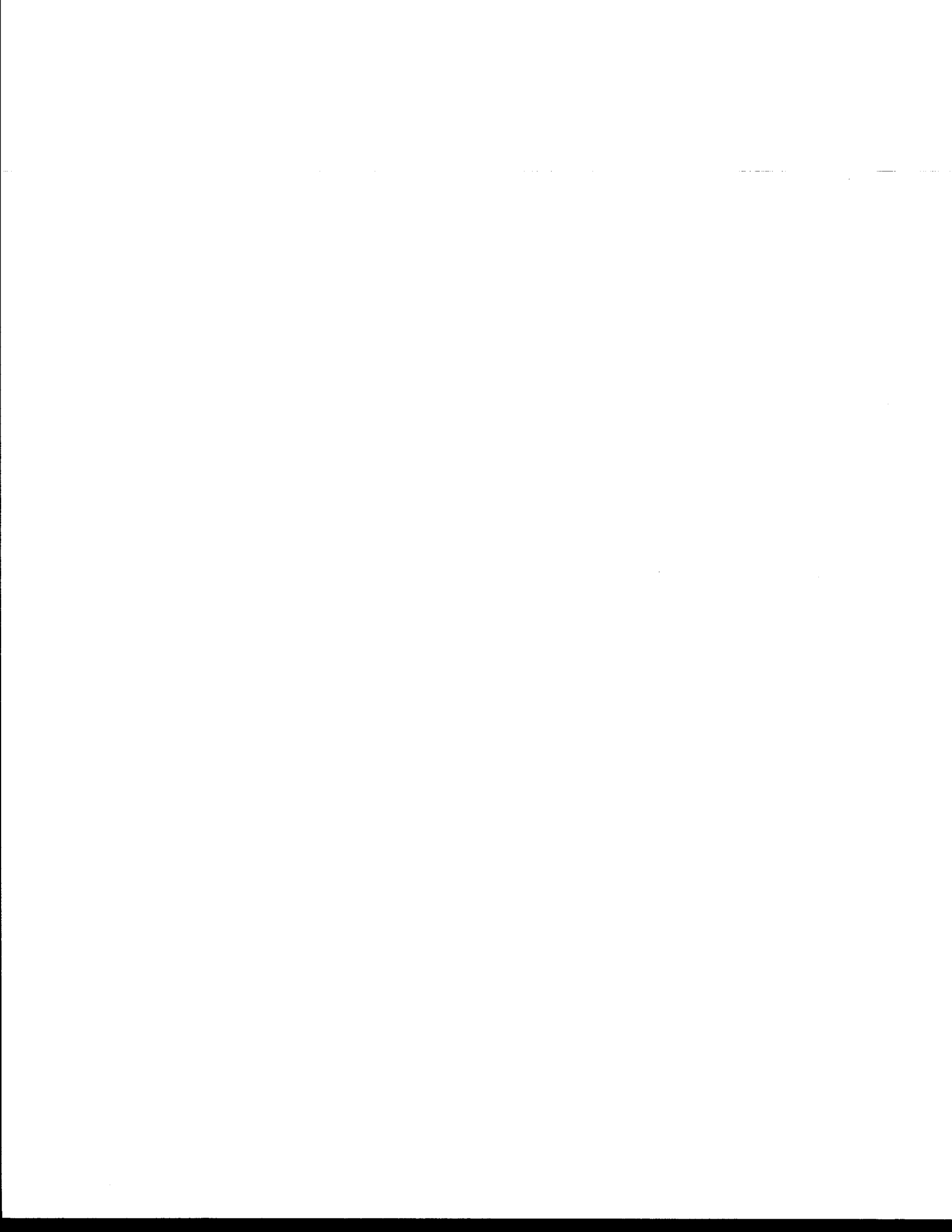


Figure A1. Van der Poel Nomograph (After Reference 22).

APPENDIX B

**Sample Calculations
for Octahedral Shear Stress Ratio**



SAMPLE CALCULATION FOR OCTAHEDRAL SHEAR STRESS RATIO

The following example problems are presented strictly as a guide to assist the user of this report with stepwise calculation of the octahedral shear stress ratio (OSR).

Stress analysis of a pavement section, as shown in Figure B1, is performed using Modified ILLIPAVE finite element computer program. However, researchers at Texas Transportation Institute (TTI) have recently developed an interactive finite element computer program (TTI-PAVE) which includes a provision to accommodate horizontal surface force in combination with the vertical load at the surface.

TTI-PAVE computer program is designed to function and perform stress analysis of pavement section on any IBM-PC compatible with the following minimum configuration:

1. 640 K of Random Access Memory (RAM)
2. Numerical Coprocessor
3. Enhanced Graphics Adaptor (EGA)
4. Enhanced Graphics Adaptor Card

However, any appropriate computer program may be used to perform stress analysis of pavement structure to obtain required information needed for calculation of octahedral shear stress ratio (OSR). After obtaining the stresses induced in the pavement due to application of loads at the surface, Equations (7) and (8) can be used to calculate octahedral normal stress and octahedral shear stress, respectively. Since the level and position of octahedral shear stress varies, depending upon pavement geometry, loading, and boundary conditions, it is advisable to calculate these values at several points within the pavement under and away from the wheel load centerline at different pavement depths.

Once the above task is performed, the octahedral shear strength of the paving mixture is obtained using the methodology outlined in this report. The steps follow:

Intermediate Overlay

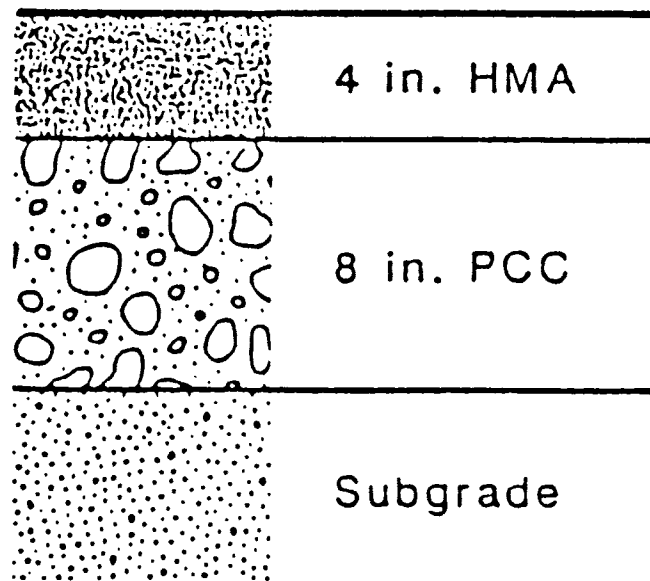


Figure B1. Pavement structure and overlay thickness evaluated on rigid base (After Reference 5).

1. Determine C (cohesion) and the ϕ (angle of internal friction) from triaxial shear test or any other compatible test method described in this report. These parameters should be obtained at the temperature and the loading rate that best simulate field conditions.
2. Determine σ_1 values concomitant with assumed σ_3 values using equation (2) or construct Mohr-Coulomb failure envelop using the C and the ϕ parameters determined in step 1.
3. Compute octahedral normal and shear stresses corresponding to failure condition of Mohr-Coulomb envelope obtained in step 2 as follows:

$$\sigma_{\text{oct}i} = 1/3 (\sigma_{1i} + 2\sigma_{3i})$$

$$\tau_{\text{oct}i} = 0.471(\sigma_{1i} - \sigma_{3i})$$

4. Construct octahedral failure envelope from $\sigma_{\text{oct}i}$ and $\tau_{\text{oct}i}$ values computed in step 3. (see Figure B2)
5. Measure C' and ϕ' (octahedral cohesion and octahedral frictional angle parameters), respectively. (see Figure B2)
6. Using these parameters (from step 5) and the normal octahedral stress value obtained from computer output (results) corresponding to the maximum octahedral shear stress induced, the octahedral shear strength of the mixture is then calculated using equation (11) as follows:

$$(\tau_{\text{oct}})_{\text{critical}} = C' + \sigma_{\text{oct}} \tan \phi'$$

7. The octahedral shear stress ratio (OSR) is thus, $\tau_{\text{oct}}/(\tau_{\text{oct}})_{\text{critical}}$, where τ_{oct} is the induced maximum octahedral shear stress within the asphalt layer as determined from analytical analysis of the pavement, and $(\tau_{\text{oct}})_{\text{critical}}$ is the maximum octahedral shear strength as computed in step 6.

SAMPLE CALCULATION

Assume a paving mixture of asphalt concrete surface layer that is extruded from the pavement will yeild a cohesive strength of 90 psi and an angle of internal friction of 35° in a triaxial shear test, as shown in Figure B2.

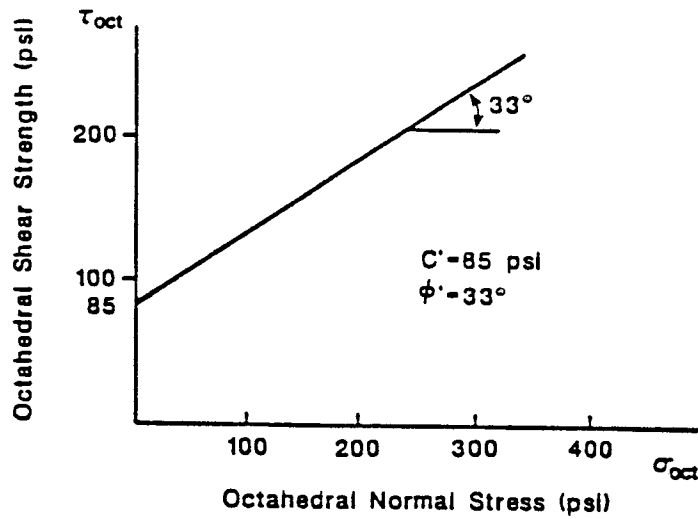
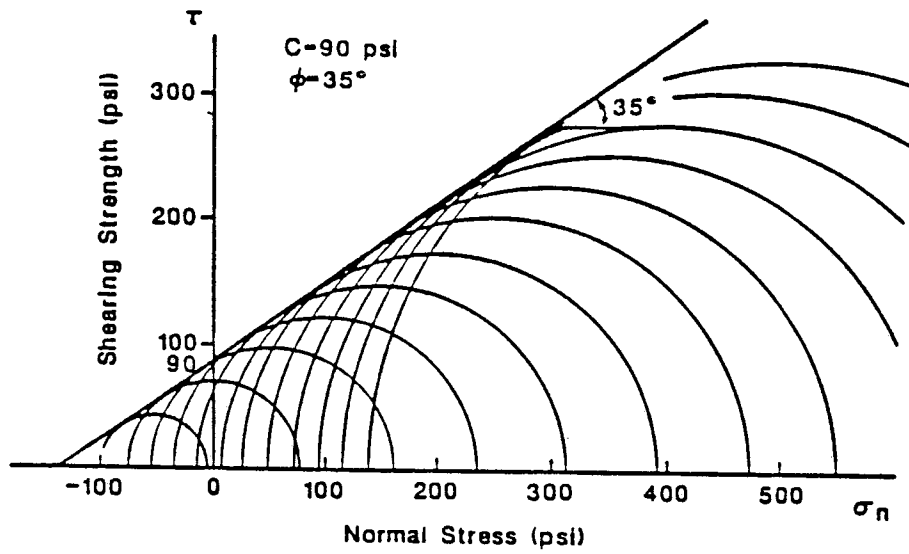


Figure B2. Typical octahedral failure envelope developed from Mohr-Coulomb failure envelope (After Reference 5).

1. Construct Mohr-Coulomb failure envelope for the C and the ϕ values assumed above (Figure B2).
2. For any Mohr's circle tangent to the Mohr-Coulomb failure envelope, the octahedral normal stress and the corresponding octahedral shear stress at failure is calculated as follows: (see Figure B2):

$$(\sigma_{octi})_{failure} = 1/3 (\sigma_{1i} + 2\sigma_{3i})$$

$$(\tau_{octi})_{failure} = 0.471 (\sigma_1 - \sigma_3)$$

3. Connect the locus of all points representing octahedral normal and shear stresses at failure (from step 2) to obtain octahedral failure envelope (see Figure B2).
4. From the octahedral failure envelope (step 3) measure C' and ϕ' . (In this example $C' = 85$ psi and $\phi' = 33^\circ$). Using equation (11), octahedral shear strength is calculated as follows:

$$(\tau_{oct}) = 85 + \sigma_{oct} \tan (33^\circ).$$

In the above expression, σ_{oct} corresponds to the induced $(\tau_{oct})_{max}$ obtained from the analytical stress analysis of the pavement, using Modified ILLIPAVE computer program.

For a 4-inch thick overlay with resilient modulus of 100 ksi and 500 ksi, respectively, over an 8-inch thick portland cement concrete (PCC) base layer with modulus of 3000 ksi resting on a clay subgrade with reactive modulus of 7500 psi, the induced critical octahedral normal and shear stresses are found for two pavement boundary conditions and are tabulated in Table B1:

5. Using equation (11) (step 4) with σ_{oct} from Table B1 for complete slippage without surface shear,

$$(\tau_{oct})_{critical} = 85 + (37) \tan(33) = 109$$

6. The octahedral shear stress ratio is then calculated as follows:

$$OSR = \frac{49}{109} = 0.449$$

Table B1. Results of Sample Calculations for a Given Pavement with a 4-inch Asphalt Concrete Surface Layer Having a Modulus of 100 ksi or 500 ksi.

Modulus of Asphalt Concrete Surface, ksi	Complete Slippage between the surface layer and the base layer				Full friction between the surface layer and the base layer			
	<u>No surface shear</u>		<u>with surface shear</u>		<u>No surface shear</u>		<u>with surface shear</u>	
	σ_{oct}	τ_{oct}	σ_{oct}	τ_{oct}	σ_{oct}	τ_{oct}	σ_{oct}	τ_{oct}
100	37	49	69	100	58	29	115	68
500	29	54	47	104	62	24	71	63