MASTER'S THESIS

M-960

SMITH, Charles William.

STRENGTH CHARACTERISTICS OF ASBESTOS MODIFIED
ASPHALT CONCRETE USING UNCONFINED TENSION
AND COMPRESSION TESTS.

Carleton University, M. Eng., 1966
Engineering, civil

University Microfilms, Inc., Ann Arbor, Michigan

STRENGTH CHARACTERISTICS OF
ASBESTOS MODIFIED ASPHALT CONCRETE
USING UNCONFINED TENSION AND
COMPRESSION TESTS

by

C. WILLIAM SMITH, B. Eng.

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR
THE DEGREE OF MASTER OF ENGINEERING

CARLETON UNIVERSITY

OTTAWA, ONTARIO

MAY, 1966

The undersigned hereby recommend to the Faculty of
Graduate Studies acceptance of this thesis,
submitted by C. William Smith, B. Eng., in partial
fulfilment of the requirements for the degree of
Master of Engineering.

.....
Dean, Faculty of Engineering

.....
Supervisor

.....
External Examiner

ABSTRACT

Increasing demands for better performance from flexible highway pavements require a re-evaluation of present design methods with the objective of utilizing a more rational approach to the problem.

This investigation was concerned with obtaining information of a fundamental nature regarding strength properties of an asphalt concrete through use of unconfined tension and compression tests. The influence on these strength properties of asbestos modification plus some comparisons with conventional design methods were significant features of the project.

Tensile and compressive strengths of specially prepared specimens were used to evaluate cohesion and angle of internal friction as defined by the Mohr failure theory. The results show that the tension-compression test has some definite promise in designing asphalt or asbestos-asphalt concrete mixtures but further work is required to fully assess its applicability. For the materials used, asbestos modification resulted in some increase in tensile and compressive strength but no significant improvement of bearing capacity. This does not preclude the possibility of significant beneficial

effects on durability through the incorporation of asbestos fibres in the mix.

The principal recommendations are concerned with establishing a continuing program to investigate more test conditions and materials and correlate the findings with conventional design methods and actual field performance.

ACKNOWLEDGEMENTS

The author wishes to express his sincere appreciation to the following people for their assistance in conducting the investigation and in preparation of this thesis:

R.C.G. Haas, Assistant Professor of Civil Engineering, for his guidance, advice, and criticism in carrying out the testing program and in preparation of this report.

N.E. Laycraft, Chief of Public Works Testing Laboratories Division, for his cooperation and assistance in providing facilities, materials, and staff for the test program.

Clark MacPhee and other staff of Public Works Testing Laboratories for their cooperation and technical assistance in preparing materials and test specimens.

L. Rice, I. Hajdu, and other staff of the Carleton University Engineering Workshop for their craftsmanship in the construction of special equipment for the test program.

The author is also deeply indebted to his wife for her patient assistance in typing the manuscript and for her continued moral support during the entire program.

TABLE OF CONTENTS

ABSTRACT	i
ACKNOWLEDGEMENTS	ii
LIST OF TABLES	vi
LIST OF FIGURES	vii
LIST OF PLATES	ix
LIST OF SYMBOLS	x

CHAPTER I INTRODUCTION

1:1	General	1
1:2	Asbestos Modification	3
1:3	A Rational Approach	3
1:4	Purpose of the Investigation and its Limitations	5
1:5	Organization of the Thesis.	7

CHAPTER II DESIGN OF BITUMINOUS PAVING MIXTURES

2:1	General	8
2:2	Determination of C and ϕ	9
2:3	Development of Bearing Capacity Equations	10
2:4	Comments on Bearing Capacity.	15
2:5	Effects and Advantages of Asbestos Modification.	18
2:6	Mix Design of Asbestos Modified Asphalts.	19

CHAPTER III TEST PROGRAM, EQUIPMENT, AND PROCEDURES

3:1	Scope of Test Program	21
3:2	Equipment	23
3:3	Materials	28
3:4	Preliminary Testing	30
3:5	Test Procedures	32

CHAPTER IV SUMMARY OF RESULTS

4:1 Description of Tabulated and Plotted
Results 38

CHAPTER V DISCUSSION AND INTERPRETATION OF RESULTS

5:1	General	60
5:2	Compressive Strength.	60
5:3	Tensile Strength.	63
5:4	Density	64
5:5	Cohesion and Angle of Internal Friction.	65
5:6	Bearing Capacity.	66
5:7	Strain at Failure	70
5:8	Voids in Mineral Aggregate and Air Voids	71
5:9	Comparison with Triaxial Results. . .	72
5:10	Results of Marshall Tests	74
5:11	Application of Tension-Compression Method to Asbestos Modified Mixes . .	76

CHAPTER VI CONCLUSIONS AND RECOMMENDATIONS

6:1	General	79
6:2	Conclusions	80
6:3	Recommendations	82

REFERENCES	85
----------------------	----

APPENDIX A Drawings of Hewitt Molds.	A1
--	----

APPENDIX B Properties of Mineral Aggregate . .	B1
--	----

APPENDIX C Typical Tinius-Olsen Charts	C1
--	----

APPENDIX D Computer Program and Results. . . .	D1
--	----

LIST OF TABLES

<u>TABLE</u>		<u>PAGE</u>
4-1	Summary of Tension-Compression Test Results	41
4-2	Summary of Triaxial Test Results	42
4-3	Summary of Marshall Test Properties	43

LIST OF FIGURES

<u>FIGURE</u>		<u>PAGE</u>
2-1	Fundamental Strength Relationships Demonstrated by the Mohr Diagram	11
2-2	Mohr Diagram for Unconfined Tension and Compression Tests	11
2-3	Stresses on an Element of Flexible Pavement Beneath a Wheel Load	13
2-4	Trigonometrical Relationships in the Mohr Diagram	13
2-5	Mohr Diagram for Unconfined Compressive Strength	14
2-6	Mohr Diagram Showing Lateral Support Equal to Unconfined Compressive Strength	14
3-1	Combinations of Asphalt and Asbestos Contents Tested	22
4-1	Compressive Strength Versus Asphalt Content	44
4-2	Tensile Strength Versus Asphalt Content	45
4-3	Compressive Strength Versus Asbestos Content	46
4-4	Tensile Strength Versus Asbestos Content	47
4-5	Density of Compressive Samples	48
4-6	Cohesion Versus Asphalt Content	49
4-7	Angle of Internal Friction Versus Asphalt Content	50
4-8	Cohesion and Friction Angle Versus Asbestos Content	51

LIST OF FIGURES (CONTINUED)

<u>FIGURE</u>		<u>PAGE</u>
4-9	Effects of Asbestos Content on C and ϕ	52
4-10	Effects of Asphalt Content on C and ϕ	53
4-11	Strain at Failure Versus Asphalt Content	54
4-12	Bearing Capacity Versus Asphalt Content	55
4-13	Contours of Constant Bearing Capacity	56
4-14	Voids in Mineral Aggregate and Air Voids in Compacted Mix	57
4-15	Comparison of Mohr Envelopes	58
4-16	Marshall Test Properties of Plain Mix	59

LIST OF PLATES

<u>PLATE</u>		<u>PAGE</u>
1(A)	Mixing Equipment	24
1(B)	Hot Water Bath	24
2(A)	Molds and Hammers	25
2(B)	Compaction Equipment	25
3(A)	Compression Testing	26
3(B)	Tension Testing	26

LIST OF SYMBOLS

γ	Density, Grams per Cubic Centimeter
P	Stress Normal to a Shear Plane
P_s	Contact Pressure Beneath a Pneumatic Tire
P_z	Normal Stress at Depth Z
P_c	Average Stress at Failure in Compression
P_t	Average Stress at Failure in Tension
S	Shear Strength as Defined by Mohr Envelope
C	Cohesion as Defined by Shear Strength at Zero Normal Stress
ϕ	Angle of Internal Friction, in Degrees, ϕ equals the slope of the Mohr Failure Line
ϵ_f	Strain at Failure, Inches per Inch

CHAPTER I

INTRODUCTION

1:1 General

Over ninety-five percent of Canada's 80,000 miles of paved roads, streets, and highways are of flexible design; that is, they are comprised of uncemented or stabilized granular materials with bituminous surfaces. Properly designed and constructed, these pavements will provide structurally sound, long lasting, economic and smooth riding thoroughfares. Much research activity has been directed to the problems of distress in these pavements, such as cracking, rutting, ravelling and other forms of disintegration. However, it is far from complete; in fact, the problem of pavement distress is being accentuated by the development of high performance vehicles with greater gross weights and higher tire pressures. Many existing pavements are not designed for such loads and pressures and certain current design methods are not

satisfactory in providing pavements to meet these demands. Some present specifications (19, p. 942)* have as governing criteria the provision of satisfactory service under conditions of 22,500 pound wheel load with a 500 pound per square inch tire pressure.

The solution to this problem lies, in part, in two areas: (a) A better understanding of the fundamental strength properties of asphaltic concrete so that rational design methods may be used and, (b) Improvement of materials and construction methods.

Many varying conditions are encountered in highway construction with respect to subgrade soils and construction materials. This has led to design procedures based on empirical tests and arbitrary limits established by the experience of engineers responsible for their development. Bridging the gap between empirical design and satisfactory field performance is an area of uncertainty reflected in "over-design" or "under-design" practices and unknown factors of safety.

* Numbers in brackets refer to literature cited. An alphabetical listing by author's name of referenced literature is to be found on 85 of this thesis.

1:2 Asbestos Modification

In the search for methods of upgrading pavement performance it has been found that the addition of a small quantity of short asbestos fibres to a bituminous mixture will, in many cases, affect the product beneficially (3, 9, 16).

The quality of a flexible pavement can be roughly assessed in terms of two factors: (a) stability, and (b) durability. The effects of asbestos on the stability of an asphaltic concrete are not clearly known but there is generally wide agreement concerning the positive beneficial effects on durability.

Experience in the design of asbestos-asphaltic concretes (9) has shown that conventional design methods, in particular the Marshall Method (1), are not entirely satisfactory. This points out the need for revised design procedures, especially those incorporating an objective, rational approach.

1:3 A Rational Approach

The need for a rational design method based on unit strength is well documented and considerable research has been conducted in this area (4, 8, 11, 12, 15).

The critical physical characteristic governing stability appears to be the shear strength, which may be expressed in terms of cohesion, C , and angle of internal friction, ϕ , as defined by the Mohr strength theory. These elements of shear strength can be determined by reproducible laboratory procedures such as triaxial tests.

Many excellent pavements have been constructed using empirical tests. One may argue that since the ultimate test of a paving mixture is its in-service performance, these "satisfactory" procedures have been justified. However, rather than indicating a vote of confidence, this may, on the other hand, indicate the lack of a quick, simple, economical, and reproducible test to evaluate shear strength properties on a more fundamental basis.

One such method, the triaxial test, was developed primarily for use in soil mechanics but was quickly found adaptable to testing bituminous mixtures (4, 15). It can give good results and is relatively simple, yet it is slow and comparatively expensive. It has, therefore, never found acceptance for routine mix design and is considered by many to be suitable only for research work (4).

Consequently, efforts have continued to devise a

method that will evaluate C and ϕ without the disadvantages of triaxial testing. Of interest in this regard is a simple tension-compression test which was investigated firstly by Hargett and Johnson (6), and later by Hewitt and Harkenhoff (7).

1:4 Purpose of the Investigation and its Limitations

The prime objective of this investigation was to determine the feasibility of applying fundamental strength properties, as determined by simple tension-compression testing, to the selection of bituminous mix design criteria. It was particularly desired to determine if this method could be applied to the design of asbestos modified mixes. The Mohr theory was used as the basis of strength evaluation.

When originally conceived the program was to have been conducted in parallel with a similar program (by the Testing Laboratories of Public Works) investigating Marshall properties of the same aggregate-asbestos-asphalt combinations. At the time of preparation, correlations between the Marshall results and compression-tension results are not possible because of an unforeseen delay in Marshall testing.

The number of variables to be considered was kept

to a minimum; only asphalt content and asbestos content were varied. All other conditions were, as far as possible, kept constant throughout the test program.

The program as conducted was further limited in that only one type of gradation of aggregate was used, only one grade of asphalt cement was used and only one set of test conditions was fully investigated; however it was considered more desirable to obtain information as definitive as possible for a limited range of conditions than to obtain inconclusive information on a wide variety of variables.

The test conditions were chosen to be the same as the Marshall Method. While these may not necessarily be optimum in nature, they were necessary for correlations since Marshall test results are meaningful only for a certain test temperature and rate of deformation.

This aspect of the program will be given further consideration in Chapter II of this thesis.

1:5 Organization of the Thesis

This thesis is presented in six chapters and appropriate appendices. The first chapter introduces

the problem and makes general observations concerning the direction and scope of the project. The second chapter presents a resume of the literature review and the development of the particular theories to be used or investigated.

Chapter three contains information concerning the test materials, procedures, and equipment used.

The results of the test program are summarized in chapter four in tabular and graphical form. A discussion and interpretation of these results follows in chapter five.

The conclusions drawn from the results and recommendations with regard to further investigation are presented in the sixth chapter.

The Appendices include detailed information pertinent to the investigation but not included in the body of the thesis, such as the equipment and material specifications. In particular, a listing of the Fortran computer program and a complete print-out of test data is presented in Appendix D.

CHAPTER II

DESIGN OF BITUMINOUS PAVING MIXTURES

2:1 General

Stability of an asphaltic concrete may be qualitatively defined as its resistance to distortion and displacement when subjected to load.

Durability of a pavement is a measure of its in-service performance, particularly with respect to the quality of the riding surface (i.e., cracking, ravelling, etc). It is more dependent on the separate physical and chemical characteristics of the asphalt and aggregate than on their combined properties in the mix. Durability may be provided by judicious selection of materials and proper construction, whereas stability is more a function of mix design.

The property of a bituminous mixture most intimately related to its stability is its shear strength. Shear strength may be represented by cohesion and angle of internal friction according to the Mohr strength theory. Figure 2-1 shows this in graphical

form, where the internal stability is represented by points along the Mohr rupture envelope. If this strength envelope is a straight line it can be represented by the general Coulomb equation:

$$S = C + P \tan \phi \quad (A)$$

where S is the shear strength

C is the cohesion

P is the normal stress on the shear plane

ϕ is the angle of internal friction

The Coulomb equation, which is predicated on the assumption that the internal resistance of materials is a function of shearing resistance due to internal friction, represents the strength at failure while the Mohr envelope represents the stress at failure. McLeod (13) has shown these to be coincident.

2:2 Determination of C and ϕ

Triaxial test methods that are used to determine C and ϕ for soils are adaptable to asphaltic concrete but are relatively cumbersome and expensive. Figure 2-2 shows graphically the principles underlying the theory proposed by Hargett and Johnson (6) to determine C and ϕ by means of simple unconfined tension and compression tests.

From the geometry of the diagram it may be derived that:

$$P_c = 2C \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \quad (B)$$

$$P_t = 2C \sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}}$$

Solving for C and ϕ

$$C = \frac{1}{2} (P_c \times P_t)^{\frac{1}{2}} \quad (C)$$

$$\phi = \sin^{-1} \frac{(P_c - P_t)}{(P_c + P_t)}$$

These equations may then be used to express unit stresses in tension and compression in terms of the strength coefficients C and ϕ .

2:3 Development of Bearing Capacity Equations

To evaluate stability of asphaltic concrete in quantitative terms two basic approaches have been used, both with the common feature of assessing the load supporting capacity of the pavement.

Hewitt (8) has used classical soil mechanics to present a strength equation of the form:

$$P_z = \gamma Z \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^2 + \frac{4C}{1 - \sin \phi} \sqrt{\frac{1 + \sin \phi}{1 - \sin \phi}} \quad (D)$$

where P_z is the vertical stress at impending failure on a triaxially loaded element, at a depth "Z", below a

FIGURE 2-1 FUNDAMENTAL STRENGTH RELATIONSHIPS
DEMONSTRATED BY THE MOHR DIAGRAM

FIGURE 2-2 MOHR DIAGRAM FOR UNCONFINED TENSION
AND COMPRESSION TESTS AT FAILURE

circular loaded area on the surface of a soil (or, in this case, a bituminous pavement) of density " γ ". This is shown graphically in Fig. 2-3 where " P_s " is the critical pressure on the surface. When considering the surface course of a pavement design, $Z = 0$ and

$$P_z = \frac{4C}{1 - \sin \phi} \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^{\frac{1}{2}} \quad (E)$$

McLeod (11) has approached the problem through an analysis of the Mohr diagram (Figure 2-4) representing failure condition of a triaxially loaded element. The following general equation is developed from the geometry of the figure:

$$\tan \phi = \frac{\frac{V-L}{2} \cos \phi - C}{L + \frac{V-L}{2} - \frac{V-L}{2} \sin \phi} \quad (F)$$

Where V is the vertical stress causing failure under a lateral confining pressure, L . Manipulation of terms gives:

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

If L is considered to be the amount of lateral confining pressure on an element of pavement below a wheel load, it has been shown, again by McLeod (11), that its maximum value at failure is equal to the unconfined compressive strength of the pavement as shown in Figure 2-6.

FIGURE 2-3 STRESSES ON AN ELEMENT OF FLEXIBLE PAVEMENT BELOW A WHEEL LOAD

FIGURE 2-4 TRIGONOMETRICAL RELATIONSHIPS IN THE MOHR DIAGRAM

FIGURE 2-5 MOHR DIAGRAM FOR UNCONFINED
COMPRESSIVE STRENGTH

FIGURE 2-6 MOHR DIAGRAM SHOWING LATERAL SUPPORT
EQUAL TO UNCONFINED COMPRESSIVE
STRENGTH

Inspection of Figure 2-5 shows that the unconfined compressive strength may be defined:

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

Substituting (H) into (G), the expression for V when the lateral stress is, in fact, as defined in equation (H), is:

$$V = \frac{4C}{(1 - \sin \phi)} \left[\frac{1 + \sin \phi}{1 - \sin \phi} \right]^{\frac{1}{2}} \quad (J)$$

which is exactly the same equation (E) proposed by Hewitt. This represents the impending failure condition; hence V (or P_z) is the ultimate "bearing capacity".

2:4 Comments on Bearing Capacity

Certain inherent assumptions and limitations exist in the determination of bearing capacity by the aforementioned methods. The following comments are therefore considered apropos:

- (1) The foregoing equation for bearing capacity is based on the following assumptions: (a) The lateral restraining pressure is equal to the unconfined compressive strength of the pavement, and (b) the medium is incompressible (i.e., Poisson's ratio, μ , = 0.5).

McLeod (11, 12) suggests that the assumption regarding lateral pressure may be conservative due to a number of factors. These include friction and stress distribution along the failure plane, except near the edge of the pavement where lateral support may approach zero.

Herkenhoff (7), however, suggests that the lateral restraint is somewhat less than the unconfined compressive strength on the basis of elastic theory. Peattie, Papazian, Subbaraju and others (19) have studied the elastic and visco-elastic properties of bituminous mixes and offer a wide variety of conclusions with respect to the elastic modulus and Poisson's ratio of asphalt concrete. Papazian's presentation of a time-frequency dependent complex modulus of elasticity and Poisson's ratio would tend to confirm McLeod's (11) conclusion that the degree of lateral support is difficult to assess and is not simply defined.

(2) Vertical stresses imposed on a pavement by pneumatic tires can be greater than the nominal pressure in the tire (12). Barber (2) shows that shear stresses in a static, pneumatic tire will increase the effective normal stress on the pavement by approximately 25 percent over the tire

pressure. He also shows that in the case of an accelerating vehicle, if the coefficient of friction between the tire and the pavement approaches 0.8, the bearing capacity of the pavement is reduced by a factor of over 4.

(3) Cohesion and internal friction are both functions of the viscous properties of the asphalt cement under any test conditions that mobilize the rheological forces in the binder. Therefore, bearing capacities as calculated from C and ϕ values that are biased by the viscous forces, are only for field conditions that approximate the loading conditions used in the laboratory.

(4) Factors involving elastic moduli of the pavement and subgrade, as investigated by Burmister (19) and others, utilizing two and three layered systems, deformations of the pavement under heavy load, and the effects of load repetition are not considered.

(5) Because of the extremely complex nature of the pavement system, bearing capacity per se is only a first approximation to rational design.

2:5.1. Effects and Advantages of Asbestos Modification

In the design of bituminous pavements one of the criteria determining optimum asphalt content is the volume of air voids in the compacted mix. If the design is too "rich" the air void content will be too low and post construction compaction due to traffic may result in the very undesirable situation of the binder "flushing" to the surface. Prime effects of such flushing are loss of stability and frictional resistance to sliding. On the other hand, high asphalt contents are desirable, within limits, to improve durability in that thicker films of asphalt surrounding the aggregate particles provide more cohesion, greater flexural strength and improved fatigue strength (16, 18). Keitzman, et al (9), have shown that the addition of short asbestos fibres can permit increases in asphalt content of up to 50 percent, and, in some cases, as high as 60 percent over optimum asphalt content as determined by the Marshall Method for the same aggregate, while preserving the stability and improving the durability of the pavement. Whereas 3 percent air voids is considered minimum by most specifications based on the Marshall Method, asbestos modified mixes are giving excellent field service with air void contents of one and two percent (3). The combined effects of low air voids and thicker binder

films delay hardening of the asphalt due to weathering and can thereby significantly contribute to the durability of the pavement (17).

2:6 Mix Design of Asbestos Modified Asphalts

The properties of asbestos modified bituminous mixes do not conform to standard mix design criteria currently in use (9). That is, criteria with respect to air voids are not necessarily satisfied and prediction of superior service at higher asphalt contents is not evident from design curves.

Keitzman, et. al. (9), recommend a compromise solution where the mix is designed without asbestos by the Marshall or some other conventional method. The asphalt content is then increased by at least 35 percent over the previously determined optimum amount. (Zuehlke (18) suggests optimum +2 percent). Asbestos fibres are then added to satisfy a critical asbestos-asphalt ratio. They suggest 0.30, 0.25, and 0.20 for heavy, medium, and light traffic respectively.

These are empirical procedures throughout but experience in the use of asbestos modified asphalt concretes would indicate that the design of these mixes

is less critical with respect to air voids and asphalt content. Because asbestos modification substantially increases the cost of materials it is important to establish suitable procedures and criteria for the optimization of the design of these mixes.

CHAPTER III
TEST PROGRAM, EQUIPMENT,
MATERIALS AND PROCEDURES

3:1 Scope of Test Program

The experimental program was designed to determine strength characteristics of bituminous mixtures within the limitations of the principle objectives of the investigation as outlined in Chapter I.

The specific variables were asphalt content and asbestos content of the trial mixes, with the asbestos content being the variable of greatest interest. Figure 3-1 shows the combinations of asbestos fibre content and asphalt cement content for which specimens were fabricated and strength tests conducted.

Asphalt cement content was varied in the unmodified mixes from 4.5 to 8 percent by weight of dry aggregate in increments of 0.5 percent, omitting 7.5 percent. Asbestos content was varied from 0 percent (i.e., the unmodified mix) to 6 percent by weight of dry granular aggregate in increments of 2 percent.

Three specimens for tensile tests and three for compression tests were molded for each test condition, except as noted in Figure 3-1, for a total of 130 specimens.

The compacted specimens were stressed to failure, in the main test program, at a temperature of 140°F and a constant deformation rate of 2.0 inches per minute (i.e., 40 percent per minute in compression tests). In addition, four particular combinations of asbestos and asphalt content were tested at 70°F and 0.045 inches per minute (i.e., 1 percent per minute) to check the method against the results of a limited triaxial test program being conducted concurrently at Carleton University (refer to D. Hayley, unpublished B. Eng. Project Report, April 1966).

% ASBESTOS							<u>TESTS</u>
	4.5	5.0	5.5	6.0	6.5	7.0	
6		1		1		1	
4		1		123		123	1
2		1		1		1	1
0	14	14	14	1234	14	1234	1
	4.5	5.0	5.5	6.0	6.5	7.0	8.0
	% ASPHALT						

1. Ten-Com 140°F,
2.0"/min
2. Ten-Com 70°F,
.045"/min
3. Triaxial 70°F,
.045"/min
4. Std Marshall

FIGURE 3-1

COMBINATIONS OF ASPHALT AND ASBESTOS CONTENTS
TESTED

3:2 Equipment

3:2:1 General

The equipment used to manufacture the briquets, except for the compaction molds, was standard asphalt laboratory equipment as used by the Department of Public Works Test Laboratories where the work was conducted. Plate 1(A) shows the mixing equipment, including the Hobart mixer, and some of the hand tools.

3:2:2 Compaction Molds

The molds used for compaction of the mixes were fabricated in the Carleton University Engineering Workshops. The basic design was essentially that set forth by Hewitt (7). Some minor modifications were made during construction, principally to cut down on costs and to simplify fabrication. Drawings for the as-built equipment are shown in Appendix A. The collars, instead of being built slightly larger than the mold, were made to exactly the same length and width dimensions, then milled out 0.020 inches on each interior surface to facilitate removal of the specimen after compaction. Alignment holes were drilled on the edges of the molds to accomodate dowel pins inserted in the mating edges of the collars. The parts were unfinished with respect to plating or rustproofing.

(A) MIXING EQUIPMENT

(B) HOT WATER BATH

(A) MOLDS AND HAMMERS

(B) COMPACTION EQUIPMENT

(A). COMPRESSION TESTING

(B) TENSION TESTING

The dimensions of the equipment were designed to provide a sample of approximately the same weight, height and compaction surface area as that of a standard Marshall sample.

Plate 2(A) shows the compaction molds on their base plates with the collars removed.

3:2:3 Compaction Hammers

Compaction Hammers were fabricated according to the Hewitt design (as shown in Appendix A) and fitted to standard Marshall compaction hammer handles providing an 18 inch drop for a 10 pound weight. Plate 2(A) also shows the compaction hammers.

3:2:4 Compaction Block

A hardwood compaction block formerly used for hand compacting Marshall specimens was modified for use in this test program. The bracket used to hold the Marshall Molds was removed and replaced with a bracket designed to hold down either the compression mold or the tension mold by means of a swinging angle iron bracket secured with a wing nut.

Plate 2(B) shows the compaction block with the compression mold secure in the bracket.

3:2:5 Tension Grips

A special set of grips was fabricated to facilitate testing the tension specimens. They conformed to the Hewitt (7) specifications but did not include provision for strain dial gage or proving ring attachments as force-deformation information was recorded directly. Drawings for the tension grips as built are included in Appendix A. Section 3:5:3 on the actual test procedures includes special notes apropos the use of the tension grips.

3:2:6 Test Machine

Strength tests on the compacted specimens were conducted on the 12,000 pound Tinius-Olsen Universal Testing machine in the Carleton University Engineering Structures Laboratory. It is fitted with an automatic recorder and appropriate strain transducers and associated accessories to record force-deformation information.

3:3 Materials

3:3:1 General

All materials used in the manufacture of the test specimens were made available through the Department of Public Works by the various suppliers.

3:3:2 Asphalt

The asphalt used throughout the entire program was penetration grade 85-100A.C., supplied by Imperial Oil Limited. The actual penetration was 94.

3:3:3 Aggregate

The aggregate was a blend of crushed stone, sand, and crusher screenings. The overall grading curve is shown in Appendix B. All aggregates were from local sources.

The sand (laboratory number 870/S) was from the Moffatt pit and composed of sub-rounded and sub-angular particles, approximately 75 percent quartz. It also included about 10 percent limestone, 10 percent granitic particles, and traces of calcite, shale, siltstone, and silty limestone.

The crushed stone (laboratory number 872/S) was from the Armstrong quarry and composed of dolomite and dolomitic limestone. The structure was fine grained and the particles were angular, generally cubicle in shape.

The crusher screenings (laboratory number 871/S) were also from the Armstrong quarry and had the same composition as the crushed stone.

Grading characteristics for each constituent and for the blend used are shown in App. B, Table B1.

3:3:4 Asbestos

Asbestos fibre used was grade 7M by the Standard Quebec Test. It is Canadian chrysotile fibre of hydrous Magnesium Silicate ($\text{Mg}_6(\text{OH})_8\text{Si}_4\text{O}_{10}$) and is a short fibre of standard production used for many years in industrial asphalts and more recently in asphalt pavements. It has a specific gravity of 2.55 and tensile strength of 100,000 to 355,000 pounds per square inch (16).

3:4 Preliminary Testing

3:4:1 General

A series of preliminary tests was conducted to familiarize the operator with the equipment and methods, to work out flaws in the equipment, and to establish the compactive effort required. Approximately 26 compression specimens and 15 tension specimens were made and subsequently stressed to failure. The Tinius-Olsen test machine was calibrated for rates of deformation of 2.0 and 0.045 inches per minute and appropriate scales were chosen for recording force and deformation.

3:4:2 Results of Preliminary Tests

It was established in the preliminary tests that the compacted density of the samples did not increase significantly with more than 75 blows on each face and that the density achieved with this compactive effort was slightly less than that achieved with 60 blows per face on the standard Marshall specimens using a mechanical compactor. However, a given compactive effort resulted in approximately equal density for compression and tension samples, all other factors remaining constant.

It was further deduced that the final compacted density was susceptible to variations in the "rhythm" of compaction. That is, the experience and the fitness of the operator had some effect on the results. As a generalization it can be stated that an operator's rhythm on a given day may begin somewhat ragged, smooth out, then deteriorate as the number of samples compacted began to induce fatigue. To reduce the effects of this factor the program was paced so that samples were constructed at the rate of about three per hour to a maximum of twelve on any one day. Also, the compression and tension molds were used alternately to reduce the effect of possibly comparing samples of unequal density, and unequal relative strengths.

It was noted during preliminary testing that the failure stress was very sensitive to density variations so this was one area in which particular care was taken.

3:5 Test Procedures

3:5:1 Material Preparation

Aggregates were separated into appropriate fractions by screening, and then recombined in the correct proportions (as per Table B1) in 1200 gram units plus the required weight of asbestos where applicable. The aggregates were maintained at 300^oF for a minimum of five hours to drive off any absorbed water and ensure even temperature distribution.

Asphalt was heated to flowing temperature and measured into cups with the weight controlled to ± 0.4 grams. Prior to use, the asphalt was held at 300^oF in an oven for one hour to achieve mixing viscosity.

A Hobart mixer was used to blend asphalt and aggregate; mixing times varied according to the ratio of asbestos fibre to asphalt. At very high ratios a very dry mix resulted which required mixing times up to two and one-half minutes. In each case about 1200 grams of the mix was used for each specimen to control

the height of the specimens to 2.50 ± 0.05 inches after compaction.

3:5:2 Compaction Procedures

The molds, when not actually in use, were stored in an oven at 175°F . During a production run they were normally used at 40 minute intervals and cleaned in varsol before being replaced in the oven. Due to the heat loss during extraction of the sample and during washing, and the relatively short periods in the oven between uses, the molds did not always regain oven temperature before re-use. However, periodic checks showed that compaction was achieved in the temperature range of $285^{\circ}\text{F} \pm 5^{\circ}$ and it was not considered that variations in the mold temperatures made any significant difference to compaction density.

To facilitate removal of the compacted briquets the molds were lightly oiled just prior to use. Rectangles of wax paper were placed above and below the unconsolidated mass to prevent adhesion of the mix to the hammers and base plates.

Compaction was achieved using 75 blows per face with a Marshall hammer modified to suit the molds used.

The compactive effort was 15 foot pounds per stroke on a sample of 0.018 cu. ft. (compacted volume) or 2250 foot-pounds per sample. This is equivalent to approximately two foot-pounds per gram as the samples averaged 1200 grams in weight.

No curing time was allowed between mixing and compacting, conforming to Marshall procedure.

The briquets were extracted from the mold by forcing them back into the collar with a hydraulic ram. Hardwood blocks of appropriate shape were inserted to facilitate the extraction as the round head of the Marshall extractor would not pass through the molds.

Weighing in air and water to determine compacted density was done only after the specimens had cooled to room temperature, generally 24 hours or more.

Compacted specimens were numbered for identification using yellow grease pencil. The four digit numbers used were coded to contain asphaltic cement content, the asbestos content, and the sample number in the particular series. Each code number occurs twice in the program, once on a compression specimen and once on the corresponding tensile specimen. In this report a C or

T subscripted to a sample number will refer to the type of failure test; briquets were not so marked but identified by shape.

For example, the code 6024T would refer to the second tensile specimen in the series 6.0% asphalt and 4% asbestos.

3:5:3 Tension and Compression Testing

Test temperature of briquets for most of the testing was 140°F and this was achieved with a control of $\pm 2^\circ\text{F}$ in a hot water bath. Samples were soaked 20 to 30 minutes prior to testing to ensure even temperature distribution.

Compressive samples were tested by placing the heated briquet on a reinforced glass plate on the lower standard of the testing machine (Plate 3(A)) and crushing it against the upper standard. The plate was inserted to protect the machine, facilitate cleanup, and yet provide a smooth non-porous surface for testing.

Tensile specimens were inserted in the tensile grips (Plate 3(B)) and pulled apart.

It was noted that the tensile specimens had no

tendency to fall out of the grips and that the face plates and their attaching hardware were unnecessary and were, in fact, not used.

All testing at 140°F was conducted at a high rate of strain, implying rapid failure, therefore it was felt that a constant temperature cabinet was not necessary, and all tests were run exposed to ambient conditions.

At the elevated test temperature there was a tendency for the tensile specimens, being tested in a vertical orientation, to fail due to self weight. This was cured by supporting the specimen with a wooden block approximately one inch thick fitted into the lower grip in such a manner that the specimen was held in a position reasonably close to the contact area of the lower tensile grip until the machine motion, i.e., the upward movement of the upper grip, reduced the slack space and initiated the testing force.

In both compression and tension testing the machine and the recording drum were preset so that a minimum amount of machine motion occurred before the specimen started to take the load. In this manner the specimens were loaded to failure within a few seconds thus preventing hardening due to excessive cooling and

consequent erroneous strength indications. The specimens were deemed to have failed when the pressure in the load cell began to fall but the load cell was not unloaded until the recording pen had run to its limit of travel in strain. A typical sample of Force-Deformation chart is included in Appendix C.

3:5:4 Deformation Rates

Rate of strain does not significantly affect the angle of internal friction (8) in triaxial testing but it does substantially change the cohesion measured at rates of strain above 0.05 inches per minute.

Because the main test program was conducted at 2.0 inches per minute to correspond with the Marshall test, it was felt that a slower rate would be desirable in the room temperature tests. A rate of 0.045 inches per minute was chosen to be in the flat range of the rate of strain versus cohesion curve, for bituminous mixtures (5) and to be compatible with rates used in Hayley's concurrent triaxial test program.

CHAPTER IV
SUMMARY OF RESULTS

4:1 Description of Tabulated and Plotted Results

A summary of compression-tension test results is tabulated in Table 4-1. Calculations of stress, cohesion, angle of internal friction, voids in mineral aggregate, and volume of air voids were made on the Carleton University IBM 1620 digital computer. A complete print-out of compiled data is included in Appendix D along with a statement listing of the Fortran II-D source program. Table 4-2 summarizes the results of the triaxial test program conducted by D. Hayley in conjunction with this project and Table 4-3 summarizes the results of the Marshall test program on the plain mix conducted by the Department of Public Works Testing Laboratories.

Figures 4-1 to 4-16 inclusive are submitted to graphically display the inter-relationships of the variables investigated.

Figures 4-1 and 4-2 show, respectively, the compressive strength and tensile strength of the specimens tested as functions of asphalt content with lines of constant asbestos content. Figures 4-3 and 4-4 show, respectively, compressive strength and tensile strength as functions of asbestos content with lines of constant asphalt contents.

Density, as a function of asphalt content, of the compression samples is shown in Figure 4-5. Lines of constant asbestos content on Figures 4-6 and 4-7 show C and ϕ as functions of asphalt content while Figure 4-8 shows both C and ϕ as functions of asbestos content for selected asphalt contents.

The effects on C and ϕ of varying asbestos content are shown in Figure 4-9 with lines of constant asphalt content. The same plot is used in Figure 4-10 to show the effects on C and ϕ of varying asphalt content at constant asbestos content. Both plots show the line of constant 1000 psi Bearing Capacity.

Figure 4-11 shows strain at failure for constant asbestos content.

Bearing capacity is shown in Figure 4-12 as a

function of asphalt content along lines of constant asbestos content and is plotted as contours on an asphalt content-asbestos content co-ordinate map in Figure 4-13.

Figure 4-14 shows air void contents and voids in mineral aggregate (V.M.A.) versus asphalt content. For purposes of clarity, lines are only drawn connecting points of zero asbestos content although points for 2, 4 and 6 percent asbestos are also plotted. The minimum limiting V.M.A. and range of permissible air voids according to specifications based on the Marshall Method (1) are also shown.

Figure 4-15 shows comparisons of Mohr envelopes for tension-compression tests of this investigation and triaxial tests of Hayley's project. All test conditions for comparison of these series were the same, such as temperature, strain rate and mixture composition. The variable then was type of test (i.e., tension-compression versus triaxial).

The results of the Marshall test program on the plain mixes is shown in Figure 4-16. Four graphs are plotted to show the effects of asphalt content on stability, density, air voids, and flow.

TABLE 4-1

SUMMARY OF TENSION-COMPRESSION TEST RESULTS

A.C.	ASB.	DENSITY	VMA	VOIDS	P _t	P _c	ε _f	C	Ø	B.C.	TEST
%	%	Gm/CC	%	%	PSI	PSI	In/In	PSI	Degrees	PSI	
4.5	0	2.296	16.6	10.6	1.8	52.4	0.016	4.8	69.0	1580	140°F
5.0	0	2.301	16.9	9.8	2.2	47.2	0.015	5.1	65.3	1041	2.0 in/min
5.5	0	2.330	16.3	8.0	2.6	47.7	0.016	5.6	63.5	913	
6.0	0	2.352	15.8	6.4	3.6	52.1	0.017	6.8	60.5	809	
6.5	0	2.352	16.2	5.8	2.5	41.4	0.016	5.1	62.2	719	
7.0	0	2.365	16.3	4.6	3.9	54.5	0.018	7.3	59.9	812	
8.0	0	2.369	16.9	3.2	5.4	50.4	0.033	8.2	53.6	516	
5.0	2	2.325	16.0	9.0	5.7	74.3	0.014	10.3	58.8	1030	
6.0	2	2.361	15.5	6.2	7.1	83.2	0.014	12.2	57.7	1047	
7.0	2	2.393	15.3	3.7	10.5	86.8	0.018	15.1	51.6	804	
8.0	2	2.398	15.8	2.1	14.5	76.0	0.029	16.6	42.7	472	
5.0	4	2.337	15.6	8.7	11.0	91.2	0.014	15.8	51.6	844	
6.0	4	2.362	15.5	6.3	11.6	98.6	0.014	16.9	52.0	935	
7.0	4	2.396	15.0	3.7	14.2	102.3	0.019	19.0	49.1	839	
8.0	4	2.398	15.8	2.3	16.5	81.5	0.030	18.3	41.4	483	
5.0	6	2.308	16.6	9.9	11.2	84.3	0.015	15.3	49.9	718	
6.0	6	2.352	15.8	6.8	16.1	96.5	0.015	19.7	45.4	673	
7.0	6	2.400	15.0	3.7	18.5	111.3	0.017	22.7	45.5	777	
8.0	6	2.381	16.4	3.1	18.1	96.3	0.022	20.8	43.0	607	0.045 in/min
6.0	0	2.334	16.5	7.2	22.0	157.2	0.013	29.4	48.9	1277	70°F
7.0	0	2.366	16.2	4.6	23.4	146.8	0.019	29.3	46.4	1066	
6.0	4	2.360	15.6	6.4	40.1	216.6	0.013	46.6	43.4	1386	
7.0	4	2.400	15.0	3.5	48.0	221.1	0.014	51.5	40.0	1239	

TABLE 4-3 SUMMARY OF MARSHALL PROPERTIES						
A.C.	DENSITY	γ	VMA	VOIDS	FLOW	STABILITY
%	PCF	Gm/CC	%	%	0.01in	Lbs
4.5	145.0	3.32	15.8	9.6	7.5	2360
5.0	146.1	2.34	15.5	8.1	7.7	2100
5.5	148.3	2.38	14.6	6.2	7.6	2340
6.0	148.5	2.38	14.9	5.4	9.4	2170
6.5	148.9	2.39	15.1	4.5	11.0	2220
7.0	148.6	2.38	15.7	4.0	12.5	1790

TENSILE STRENGTH VERSUS ASPHALT CONTENT

FIGURE 4-2

ANGLE OF INTERNAL FRICTION VERSUS ASPHALT CONTENT

FIGURE 4-7

EFFECTS OF ASBESTOS CONTENT ON C AND ϕ
 --- LINES OF CONSTANT ASPHALT CONTENT

FIGURE 4-9

EFFECTS OF ASPHALT CONTENT ON C AND ϕ
 --- LINES OF CONSTANT ASPHALT CONTENT

FIGURE 4-10

FIGURE 4-12

FIGURE 4-13

VOIDS IN MINERAL AGGREGATE
AND AIR Voids IN COMPACTED MIX

FIGURE 4-14

COMPARISON OF MOHR ENVELOPES

6% A.C.

TESTS

100 F, 2"/MIN TENS-COMP -----

70 F, .045"/MIN " " —————

70 F, .045"/MIN TRIAXIAL -----

SCALE: 1"=20 PSI

7% A.C.

FIGURE 4-15

MARSHALL TEST PROPERTIES

PLAIN MIX

FIGURE 4-16

CHAPTER V

DISCUSSION AND INTERPRETATION OF RESULTS

5:1 General

The purpose of this investigation was to examine some of the strength properties of certain bituminous mixtures as determined by the tests used in order to evaluate these tests as a means of selecting possible design criteria. This chapter discusses the findings of the program in detail and attempts to show the significance of test results and their relationships to the parameters investigated.

5:2 Compressive Strength

Failure of asphaltic concrete specimens in the compression test is a pure shear failure with well defined shear planes along the sides of wedges formed at the top and bottom of the specimens. Shear strength may be defined in terms of two parameters, cohesion and internal friction, both affected by the bituminous binder. As asphalt is added, cohesion between the particles is increased up to a maximum,

depending on the type and quality of the aggregate and the properties and distribution of the binder. At the same time, the lubricating properties of the binder cause a decrease in the friction component. Hence shear strength will have a maximum value at a certain asphalt content that gives an "optimum" combination of cohesive and frictional resistance.

The influence on unconfined compressive strength of altering the asphalt content is shown in Figures 4-1 and 403. In Figure 4-1 the lower curve representing the strength of the plain mixes is slightly concave downward and has a very flat slope. That is, increasing the asphalt content in the plain mix increased the unconfined compressive strength by only a small amount, up to about 7 percent asphalt, then caused a decline in strength. Inspection of Figures 4-5 and 4-14 will assist in explaining this behaviour. Figure 4-5 shows that the density of this mix approaches its maximum value at 7 percent asphalt and Figure 4-14 shows that the optimum required air voids are also obtained at about 7 percent asphalt. This would indicate that minimum aggregate particle spacing and low level of air voids consistent with sufficient binder to provide cohesion are the requirements for a strong asphaltic concrete.

Use of less than optimum asphalt leaves greater air voids which implies a less effective bond between particles and resulting lower cohesion. On the other hand, a greater amount of asphalt overfills the void spaces, forces the particles apart, and decreases both cohesion and friction.

Asbestos fibres added to the mix affect the compressive strength two ways. The strength is increased 50 percent to 100 percent and the peak in strength at 7 percent asphalt is accentuated. It is to be noted that the first increment of asbestos fibre has the greatest effect on the strength at any asphalt content as seen in both Figures 4-1 and 4-3. Asbestos fibres are highly absorptive, yet have very high tensile strengths, and it would appear that this substantially adds to the cohesive strength of the binder. Larger quantities of asbestos, however, do not continue to add correspondingly larger increments to the compressive strength. This suggests that the increase in cohesion is offset by greater loss of friction. This is to be expected as the fine, non-angular asbestos fibres fill the interstitial spaces between the angular aggregate particles.

5:3 Tensile Strength

Failure of the tensile specimens cannot be classified as the same type of shear failure as exhibited by the compressive specimens in that no well defined shear planes were formed. Rather, the failure occurred as a series of tension cracks propagated along the edges until, at some critical strain, the bonding failed between particles along the failure plane. That is, the relative motion occurred normal to the failure plane rather than parallel to it. This is a measure of the intrinsic strength of the asphaltic cement; not of the aggregate and binder in combination.

Figures 4-2 and 4-4 are presented to show the effects of changing asphalt and asbestos contents on tensile strength. In general, the tensile strengths followed the same trends as the compressive strengths but failed to peak at an optimum asphalt content. Also, in general, incremental changes in asbestos content, were more significant than incremental changes in asphalt content, with the first increment of asbestos being most significant. The increase in tensile strength due to asphalt can be primarily attributed to lower air voids, hence greater effective cross-sectional area. The increase due to asbestos can be attributed to the high tensile strength of the asbestos fibres.

5:4 Density

The density versus asphalt content curves presented in Figure 4-5 are those for the compression specimens and are considered, with one exception, to represent the densities of the tensile specimens with reasonable accuracy. The exception is the 5 percent asphalt, 4 percent asbestos series in which the average density was 2.40 grams per cubic centimeter for the compression specimens, whereas the corresponding tensile specimens averaged 2.33 grams per cubic centimeter. It is considered that the most probable density truly representing this condition is the lower value. It is plotted showing, with a broken line, the trend to be expected between 5 and 6 percent asphalt at the 4 percent asbestos level while the solid line shows the density trend as recorded.

It is seen that the maximum densities were achieved in the region of the 7 percent asphalt content and that the asbestos mixes, were, in general, somewhat more dense. It is further observed that, at peak density, asbestos content had practically no influence on the density.

5:5 Cohesion and Angle of Internal Friction

The relative changes in magnitude of compressive and tensile strengths and their explanation in terms of changes in cohesion and angle of internal friction are verified in the plots of Figures 4-6 and 4-7. Cohesion in the plain mix increases slowly between 4.5 and 7 percent asphalt then levels off. The very high tensile strength of the asbestos fibres and their affinity for asphaltic cement is reflected in increasing cohesion with increasing asbestos content at any asphalt content. Again the incremental change in cohesion is most pronounced with the first increment in asbestos content, but decreases at higher asbestos contents.

The angle of internal friction decreases with increasing asphalt and also with increasing asbestos content. The former action may be attributed to the increased lubrication effect while the addition of asbestos fibres, which are non-granular, tends to fill in the interstitial spaces and reduce friction.

Figure 4-8 shows the change in cohesion and friction angle at constant asphalt content due to the change in asbestos content. Again it is seen that the first increment of asbestos is the most effective in

increasing the cohesion and decreasing friction and that above 4 percent asbestos the incremental effect is considerably reduced.

This is symptomatic of a reduction in effective binder content and can probably be attributed to the high absorption of bitumen by the asbestos fibres.

Figures 4-9 and 4-10 are included to show the interaction between cohesion and angle of internal friction as asbestos content and asphalt content respectively are varied. In all cases, as seen previously, increasing asbestos content at constant asphalt content results in increased cohesion but decreased friction. Similarly, increasing asphalt content at constant asbestos content increases cohesion but decreases friction. Comparing Figures 4-9 and 4-10 it is seen that changing asphalt content has a greater effect on friction than on cohesion while changing asbestos content has a more marked effect on cohesion.

5:6 Bearing Capacity

Bearing capacities as calculated by equation (E), Chapter II, are presented in two graphs. Figure 4-12 presents bearing capacity as a function of asphalt

content for constant asbestos contents and Figure 4-13 is a plot of contours of constant bearing capacity in an asbestos content - asphalt content coordinate map.

Wide variations in the value of bearing capacity with respect to changing asbestos content is shown by Figure 4-12. The plain mix exhibits the greatest change in bearing capacity with changing asphalt content; it falls from 1580 psi at 4.5 percent asphalt to 516 psi at 8 percent, losing one-third of its strength between 4.5 and 5 percent asphalt.

On the other hand, the 6 percent asbestos mixes were almost insensitive to changes in asphalt content in the range of asphalt contents studied. Intermediate quantities of asbestos content had a peak value of bearing capacity between 5.5 and 6 percent asphalt and fell off quite appreciably at higher asphalt contents.

In general, it can be stated that neither high asbestos contents nor high asphalt contents contribute to high bearing capacities but in the range of 6 to 7 percent asphalt some benefit may be gained by the addition of 2 to 4 percent asbestos.

Relating these observations to basic strength

properties it is seen that bearing capacity is more sensitive to changes in the friction angle than to changes in cohesion.

The bearing capacity contour map in Figure 4-13 indicates a ridge of maximum bearing capacity oriented diagonally through the plot, with greatest values of bearing capacity occurring at the lowest values of the plot parameters. This trend confirms the hypothesis stated in Chapter II, para 2:4(5) that bearing capacity, per se, is not a satisfactory criterion for mix design because it does not account for the properties associated with durability. In fact, it will subsequently be shown that some of the high bearing capacity mixes did not satisfy criteria for air voids.

Discussion of previous parametric curves has indicated peaks in certain of the properties under investigation in the region of 7 percent asphalt content. Considering the bearing capacity map, it may be seen that the high strength ridge crosses the 7 percent asphalt line between 2 percent and 4 percent asbestos. This suggests that if 7 percent is, in fact, the optimum asphalt content, then 3 percent asbestos

would be the optimum asbestos content. Such a combination would be expected to give an ultimate bearing capacity of about 820 psi according to the method of testing used in this program. This will be compared with Keitzman's empirical design method in Section 5:11.

Referring back to Figures 4-9 and 4-10, and in particular the lines of constant bearing capacity, and keeping in mind that bearing capacity values increase with both cohesion and angle of friction, it is seen that lines of constant asphalt content tend to parallel the lines of constant bearing capacity except at very high asbestos content. However, lines of constant asbestos content diverge sharply from lines of constant bearing capacity, and in a direction of decreasing bearing capacity as asphalt content is increased.

This would indicate that the relationship between C and ϕ and the relative effects of asphalt on those properties are such that the loss of friction due to an increment of asphalt content is just counteracted in the bearing capacity equation by the associated increase in cohesion. Asbestos addition has relatively greater effect on friction than on cohesion, hence the

decrease in bearing capacity which is more sensitive to friction changes than cohesion changes.

5:7 Strain at Failure

The effects of asphalt and asbestos contents on the strain at failure of the compressive samples is shown in Figure 4-11. Although flexibility was not one of the properties of asphalt pavement of prime interest in this study, it may be related to strain at failure, hence these results are particularly noteworthy.

It is seen that, up to about 7 percent, asphalt content has little effect on strain at failure but at 8 percent the strain has nearly doubled. The effect of asbestos is to reduce the strain at failure or, in effect, produce a more brittle mixture. This is most noticeable at 6 percent asbestos.

In this case, the increase in strain at failure at 8 percent asphalt over that at 7 percent is only one-third of the similar increase in the plain mix. The increased strain at failure due to asphalt may be attributed to the thicker binder films between particles which would permit a larger relative linear displacement between particles for the same angular displacement. Addition of asbestos tends to reduce film thicknesses

by absorbing some of the binder and filling in more pore spaces between the aggregate particles.

5:8 Voids in Mineral Aggregate and Air Voids

The percentage of voids in the mineral aggregate (V.M.A.) is a measure of the degree of compaction and is related to particle shape and orientation and to the grading curve for the aggregate. It can be seen in Figure 4-14 that the amount of asphalt in the mix had very little effect on the final arrangement of aggregate particles. That is, with the compactive effort used the V.M.A. remained almost constant.

As would be expected, asbestos fibres tended to slightly lower the V.M.A. by filling in some of the interstitial void spaces. In most mixes the V.M.A. was within 1 percent of the minimum specified by the Marshall method using maximum sized coarse aggregate of 3/8 inches.

The lower line on Figure 4-14 shows the air voids in the compacted mixes. The continual decrease in air voids with increasing asphalt is to be expected as, at nearly constant V.M.A., the additional asphalt is simply filling in the voids. Asbestos content had

very little effect except to slightly lower the percentage of air voids at any particular asphalt content.

Marshall specifications, with respect to air voids, would, as indicated, reject all mixes with less than 6.5 percent asphalt as being too "lean" and consider 6.5 to 7.5 percent acceptable. The 8 percent mix is marginal at zero asbestos but too "rich" with asbestos added.

5:9 Comparison with Triaxial Results

The Mohr failure envelopes shown in Figure 4-15 compare the results of Hayley's triaxial tests with the results of the tension-compression tests of this investigation. The same rate of strain and test temperature was used in each instance on selected trial mixes. Also included for comparison are the failure envelopes for the same mixes tested in the main program.

With one exception (the 7 percent asphalt plain mix) values of cohesion developed in the triaxial test were 15 to 20 psi higher than those developed in tension-compression tests. Friction angles in each case are in reasonable agreement although insufficient data are

available to statistically test for the significance of the difference in angles (i.e., each line is the result of only one series of tests).

Inspection of Hayley's results in Table 4-2 shows that the value of cohesion in the 7 percent plain mix was derived with a very large possible error. It is felt that this is a result of experimental deviation rather than an anomaly in the behaviour of the mix. It is apparent that while similar trends are indicated, the tension-compression test gives substantially different values of cohesion than triaxial testing. The prime reason for this probably lies in aggregate particle orientation.

In the triaxial test the force is applied parallel to the direction in which compactive forces were imposed whereas in the tension-compression tests, as used in this program, the test force was applied perpendicular to the direction of compactive forces. Puzinauskas (14) has shown that compactive forces cause the aggregate particles to orient themselves in a direction perpendicular to the force of compaction and that, as a result, the strengths measured by loading parallel to the force of compaction are greater

than those measured when loading is perpendicular to the force of compaction. He further shows that the ratio of unconfined compressive strengths in the respective directions may range from 1.5 to 2.5 for the type of compaction used in these tests. This indicates that one may expect greater cohesion values from the triaxial test than from the tension-compression test.

5:10 Results of Marshall Tests

Marshall tests were conducted on unmodified trial mixes using asphalt contents ranging from 4.5 percent to 7 percent. It is anticipated that the Department of Public Works will resume work on the modified mixes at some future date.

The Marshall Method utilizes five empirical parameters to select optimum mix design. Four of these are presented in graphical form in Figure 4-16; the fifth, voids in the mineral aggregate, was omitted as values obtained were substantially the same as shown for tension-compression samples in Figure 4-14. They were in no case critical in the mix design of the aggregates used, insofar as design specifications are concerned.

Analysis of Figure 4-16 yields the following results:

(a) Density: Maximum density of 148.9 pounds per cubic foot (2.39 grams per cubic centimeter) occurred at 6.5 percent asphalt.

(b) Marshall Stability: Values of stability were widely scattered and did not behave in a well defined manner. However, all values were very much above the minimum of 750 pounds specified for design.

(c) Marshall Flow: The minimum specification for flow is 8 one-hundredths of an inch, which would demand a minimum asphalt content of about 5.5 percent.

(d) Air Voids: Very little scatter is evident in the plot of air voids versus asphalt content. The maximum allowable voids (5 percent) are obtained with a minimum asphalt content of 6.3 percent.

Considering the above criteria the mix of maximum density which satisfies the air voids specification would contain 6.5 percent asphalt and would be chosen for this design. This design is also acceptable according to the specifications for the other three parameters.

The results of the tension-compression tests would indicate a choice of 7 percent asphalt in the plain mix design based on maximum density, compressive strength, flexibility and air voids. This would be a slightly less economical mix than the 6.5 percent recommended by the Marshall Method.

5:11 Application of Tension-Compression Method
to Asbestos Modified Mixes

Analysis of the results of the tension-compression tests would indicate that, for the aggregate used, a modified mix containing 7 percent asphalt cement and 3 percent asbestos fibres would be optimum, based on the bearing capacity and maximum allowable air void content.

Keitzman's method, as referred to in Chapter II, would suggest an asbestos modified mix designed as follows:

- (a) Asphalt content: Marshall Method optimum for plain mix plus 35 to 50 percent. In this case, using the minimum of 35 percent this would require 9 percent asphalt to the nearest .5 percent.
- (b) Asbestos content: Use asbestos to asphalt ratio of 0.3 for heavy duty pavement, i.e., 2.5 percent asbestos fibre to nearest 0.5 percent.

Extrapolation of the results of the tension-compression testing would indicate that such a mix would be very weak in both tension and compression, would have very low cohesion and friction angle, and would have very low bearing capacity. In other words, it would likely be a very "rich" mix. On this basis it appears that the empirical modification suggested by Keitzman cannot be satisfactorily applied to designing a mix with the materials of this investigation.

The aggregate used in this program possesses characteristics which apparently can provide satisfactory pavement without asbestos modification. The incremental gains in strength in the compression and tension tests are not reflected in substantially higher bearing capacity; hence the use of asbestos modification appears to result in no particular strength advantage for the materials used. It is possible, of course, that modification could be justified on the basis of increased durability; however, the extra costs involved would have to be given careful consideration.

One may also consider the point that the decision to incorporate asbestos into asphalt mixes has been made in any case in a considerable number of actual

applications. For these instances, the engineer must produce satisfactory designs. Thus the investigation of methods such as the tension-compression test can be quite justified in this respect as the Marshall test does not yield results that satisfy present criteria.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6:1 General

This chapter summarizes the conclusions drawn from the results of the testing program and presents some suggestions for future work in the same area. .

The investigation was conducted to evaluate the tension-compression test method of asphaltic concrete mix design with emphasis on applicability to asbestos modified mixes. It can only be considered as a preliminary effort in this area and conclusions submitted must be viewed in the light of the very restricted scope of the investigation. The limitations imposed by (i) the use of only one aggregate, (ii) the investigation of only one temperature-rate of strain combination, (iii) the use of only a small number of samples at any one test condition, and (iv) the lack of sensitivity of the test equipment and apparatus, and other factors, have somewhat restricted a complete achievement of the original rather broad objectives of

the program. However, the results obtained were considered significant and should provide a basis for future work.

The following section summarizes in point form the pertinent conclusions.

6:2 Conclusions

(1) The tension-compression test method compares two somewhat different modes of failure and combines the results into a simple strength relationship expressed by the Mohr Theory. It may therefore be questioned on this basis as a true indicator of shearing strength although it may be entirely realistic in simulating the behaviour of an in-service bituminous mixture subjected to traffic loads.

(2) Values of cohesion as determined by the tension-compression tests are substantially lower than those determined in the triaxial tests; this is thought to result primarily from the orientation of aggregate particles. The phenomenon seems to have very little effect on the values of angle of internal friction.

(3) The tension-compression test appears to show some definite promise as a practical method of determining

"optimum" asphalt and asbestos contents for an asbestos modified mixture. However, as only a very limited range of conditions was investigated, considerable further work would be required to assess its true applicability.

(4) Bearing capacity alone is not a sufficient criterion for design of asphaltic concrete mixtures as it does not account for the factors affecting durability.

(5) Asbestos modification of a plain mix that exhibits good strength properties is not justified on the basis of marginal strength increments indicated by the testing methods and materials of this program. Although asbestos modification generally results in significantly higher cohesion, its effect is offset by a lower angle of internal friction. This conclusion does not obviate its possible usefulness in upgrading a mixture of inferior stability or of increasing the durability of an asphalt mixture.

(6) The use of large amounts of asbestos, (i.e., in excess of 4 percent for the mix of this investigation) may actually have a harmful influence in that the friction angle (hence the bearing capacity) is seriously reduced. For the materials and methods of this investigation, satisfactory modification was achieved by adding about 3 percent asbestos.

(7) Presently suggested mix design methods for asbestos modified asphaltic concrete should be used with extreme caution as it would appear from this investigation that they may not be applicable to a variety of conditions.

6:3 Recommendations

Several aspects of the subject area under consideration appear to warrant further investigation.

The following recommendations are submitted to offer, in very broad terms, some guidance for subsequent work.

(1) Firstly, the investigation described in this thesis cannot be considered fully completed until such time as information is available on the Marshall Method properties of the trial mixes used and until correlations are made between design parameters.

(2) It would appear that the tension-compression tests have some merit in designing asbestos modified bituminous mixes but further investigation of test conditions is needed. In particular, tests at very slow rates of deformation (0.01 inches per minute) should be carried out to more accurately assess

fundamental properties of the mix exclusive of viscosity effects that are speed and temperature sensitive.

(3) A variation of the basic test may be possible wherein a simple direct shear test is used in conjunction with an unconfined compression test to define the "C" intercept and angle " ϕ " of the Mohr rupture envelope.

(4) It has been suggested in the conclusions of the previous section that asbestos modifications of a "satisfactory" design is not warranted on the basis of these tests. It is therefore proposed that an investigation be carried out on the modification and testing by means of tension-compression tests on a mix known to have inferior qualities as determined by the Marshall Method.

(5) As yet very little is known on a rational basis of the significant factors affecting durability, particularly with regard to asbestos modification. Further work is required to assess these factors in terms of and in conjunction with strength and performance indicators.

(6) Subsequent investigations should carefully

consider particle orientation with respect to direction of compaction and load application. Also, the type of compactive equipment used (kneading, dynamic, or static) will produce different strength properties that may be evaluated by means of tension-compression tests.

(7) Future investigations may benefit by the use of strength testing equipment incorporating the following features: (i) greater sensitivity in load and strain rate, (ii) load repetition capability, and (iii) environmental chamber. This would permit more critical appraisal of strength-deformation characteristics.

(8) Finally, it is considered essential that a true and complete assessment of the method is only possible when laboratory investigations are integrated with field projects to correlate in-service performance with the results of experimental programs. In this regard, close cooperation between the University and the Federal Department of Public Works, as, experienced in this program, should be encouraged to the mutual benefit of both agencies.

REFERENCES

1. Asphalt Institute MS-2 "Mix Design Methods for Hot Mix Asphalt Concretes", 1962.
2. Barber, E. S. "Shear Loads on Pavements" Proceedings of International Conference on Structural Design of Asphalt Pavements, 1962.
3. Brooks, D. W. "Asbestos Modified Asphaltic Concrete Pavements" Unpubl term report, Carleton University, 1964.
4. Goetz, W. H. "Comparison of Triaxial and Marshall Test Results". Proc., A A P T , Vol 20, 1951.
5. Haas, R. C. G. "Shear Strengths of Sand-Asphalt Mixtures". Unpub. M.Sc. Thesis, U. of A., 1963.
6. Hargett, E. R., and Johnson, E. E. "Strength Properties of Bituminous Concrete Tested in Tension and Compression" Proc., HRB, Vol 40, 1961.
7. Herkenhoff, P. G. "A Determination of the Shear Strength Properties of Bituminous Mixtures by the Use of Tension and Unconfined Compression Tests". Proc., AAPT, Vol. 33, 1964.
8. Hewitt, W. L. "Analysis of Stresses in Flexible Pavements and Development of a Structural

- Design Procedure" HRB
Bulletin 269, 1960.
9. Keitzman, J. H.,
Blackhurst, M. W., and
Foxwell, J. A. "Performance of Asbestos-
Asphalt Pavement Surface
Courses with High Asphalt
Contents". HRB Highway
Res. Rec. 24, 1963.
10. Lefebvre, J. "Discussion on Theoretical
Specific Gravity". ASTM,
STP 191, 1956.
11. McLeod, N. W. "A Rational Approach to
the Design of Paving
Mixtures". Proc., AAPT,
Vol. 19, 1950.
12. McLeod, N. W. "Rational Design of
Bituminous Paving Mixtures
with Curved Mohr
Envelopes". Proc., AAPT,
Vol. 21, 1952.
13. McLeod, N. W. "The Mohr Diagram and
Coulomb Equation". Proc.,
HRB, 1950.
14. Puzinauskas, V. P. "Influence of Mineral
Aggregate Structure on
Properties of Asphalt
Paving Mixtures". Proc.
HRB, Jan. 1964.
15. Smith, V. R. "Triaxial Stability
Method for Flexible Pave-
ment Design". Proc.,
AAPT, Vol 18, 1949.
16. Tamburro, D. A.,
Blekicki, H. T., and
Keitzman, J. H. "The Effects of Short
Chrysotile Asbestos
Fibres on the Structural
Properties of Asphalt
Pavements". Proc., AAPT,
Vol. 31, 1962.
17. Traxler, R. N. "Durability of Asphalt
Cement". Proc., AAPT,
Vol 32, 1963.

18. Zuehlke, G. H. "Marshall and Flexural Properties of Bituminous Pavement Mixtures Containing Short Asbestos Fibres". HRB, Highway Res. Rec. 24, 1963.
19. Proceedings, International Conference on the Structural Design of Asphalt Pavements, (Ann Arbor, Michigan) 1962.

APPENDIX A

DRAWINGS OF HEWITT MOLDS

- Compression Molds
- Tension Molds
- Base Plates
- Hammers
- Tension Grips

A-1

12/2/75

NOTE: MOLD ASSEMBLY MADE WITH C.A. STEEL PLATE $\frac{3}{8}$ " THICK
THROUGHOUT THICKNESS OF FLASK 4"O ROUND STOCK

ELEVATION

PLAN

327

DECLARATION

ALL PARTS:

MATERIAL - C.R.S.
ASSEMBLY

12-10-1964

CARLETON UNIVERSITY		
SCALE HALF SIZE	HEWITT MOLDS FOR COMPACTING BITUMINOUS TEST SAMPLES	DRAWN BY C. W. SMITH
SHEET 1 OF 5		OCT 1965

SECT A-A

ASSEMBLY: $\frac{1}{8} \times \frac{1}{4}$ FLAT HEAVY SCREWS

BASE PLATES

HEWITT MOLDS
SCALE: HALF SIZE
CHAS. SMITH OCT 65

MOLD

Inside Faces of
Casing Section
Casing Section
Casing Section

5 1/2"

5 1/4"

Inside Faces of
Casing Section
Casing Section
Casing Section

Inside Faces of
Casing Section
Casing Section
Casing Section

COLLAR

WATER MOLD
SCALE 1/2" = 1" SHEET 3 OF 5
C. W. B. 100 05

MOLD

1/2"

Inside Face of
Casing Section
to be Made
in 1955-1956

1/2"

1/2"

1/2"

Inside Face of
Casing Section
to be Made
in 1955-1956

1/2" Alignment Pin

HEWITT Molds
SCALE HALF SIZE SHEET 3 OF 5
CIVS-NITH OCT 65

NOTE 1

NOTE 2

COMPACTION HAMMERS

NOTE 3

NOTE 4

NOTE 1 BORE & TAP TO RECEIVE MARSHALL HAMMER

HERNITT MOLD

NOTE 2 ASSEMBLY: 2 1/4" FLAT HEAD SCREWS

4 EACH HAMMER

SCALE: HALF SIZE SHEET 4 OF 5

CH. SMITH OCT 65

APPENDIX B

PROPERTIES OF MINERAL AGGREGATE

- Grading Curve
- Table of Physical Properties

TABLE B1

PHYSICAL PROPERTIES OF AGGREGATES

LAB. NUMBER:	807/S	871/S	872/S	MIX
% Passing:				
1/2"			100	100
3/8"	100	100	82	95
#4	94	94	3	67
#10	91	56		53
#20	84	32		44
#40	62	22		32
#80	9	17		8
#200	3	13		5
% by Weight	40	30	30	100
Specific Gravity*	2.588	2.590	2.757	2.639

* ASTM BULK Specific Gravity

APPENDIX C

TYPICAL TINIUS-OLSEN CHARTS

- Compression Test
- Tension Test

ELONGATION-COMPRESSION

MINIUS OLSEN TESTING MACHINE CO.

Test No. 10116 Size 2.51 Area 1.055 Ultimate Str. lbs. 618
 Yield Point Lbs. 54.10 Per Cent Elongation 10.00
 Date 11/2/66

Test No. 7024 T Size 2.55 Area .33 Yield Point Lbs. Sq. In. 15 Per Cent Elongation .1 Per Cent Reduced Area .1 Date 10-2-45

APPENDIX D

COMPUTER PROGRAM & RESULTS

- Fortran II-D Program
Listing
- Print-Out of
Calculations

```

C      ENG 599 (THESIS) C W SMITH      ANALYSIS OF EXPERIMENTAL DATA
DIMENSION ST(3),SC(3),NTND(3),THT(3),TPF(3),NCRU(3),CHT(3),
1CPFF(3),TDEN(3),CDEN(3),CWAG(3),TWAG(3),CVMA(3),TVMA(3),CVAV(3),
2TVAV(3),KEF(3),EF(3)
48 READ 105,ASTHb
READ 120,N1,N2
K=0
4 READ 102,AC,AS,AAB
IF (N1-K) 9,9,0
6 READ 100,NCRU(1),CHT(1),CPFF(1),CDEN(1),NCRU(2),CHT(2),CPFF(2),
1CDEN(2),NCRU(3),CHT(3),CPFF(3),CDEN(3),C,KEF(1),KEF(2),KEF(3)
60 TU 30
9 READ 109,NCRU(1),CHT(1),CPFF(1),CDEN(1),NCRU(2),CHT(2),CPFF(2),
1CDEN(2),NCRU(3),CHT(3),CPFF(3),CDEN(3),C,KEF(1),KEF(2),KEF(3)
50 READ 101,NCRU(1),THT(1),TPF(1),TDEN(1),NCRU(2),THT(2),TPF(2),
1TDEN(2),NCRU(3),THT(3),TPF(3),TDEN(3),T
DO 20 I=1,3
CWAG(I)=CDEN(I)*10000./(100.+AC)
TWAG(I)=TDEN(I)*10000./(100.+AC)
TVMA(I)=100.-TWAG(I)/ASTHb
CVMA(I)=100.-CWAG(I)/ASTHb
CVAV(I)=CVMA(I)-(CWAG(I)*(AC-AAB)/102.)
TVAV(I)=TVMA(I)-(TWAG(I)*(AC-AAB)/102.)
EF(I)=KEF(I)
ST(I)=TPF(I)/(THT(I)*2.)
20 SC(I)=CPFF(I)/(CHT(I)*2.)/(1.-EF(I)/10000.)
CVAVA=(CVAV(1)+CVAV(2)+CVAV(3))/3.
TVAVA=(TVAV(1)+TVAV(2)+TVAV(3))/3.
CVMAA=(CVMA(1)+CVMA(2)+CVMA(3))/3.
TVMAA=(TVMA(1)+TVMA(2)+TVMA(3))/3.
SIGT=(ST(1)+ST(2)+ST(3))/T
SIGC=(SC(1)+SC(2)+SC(3))/C
TDA=(TDEN(1)+TDEN(2)+TDEN(3))/3.
CDA=(CDEN(1)+CDEN(2)+CDEN(3))/3.
CU=0.5*SQRTE(SIGT*SIGC)
SIMP=(SIGC-SIGT)/(SIGC+SIGT)
S=SIMP*SIMP
PHI=300./6.283*ATANF(SQRTE(S/(1.-S)))
BC=4.0*CU/(1.0-SIMP)*SQRTE((1.+SIMP)/(1.-SIMP))
PRINT 105,AC,AS
PRINT 106
PRINT 107,NCRU(1),CHT(1),CDEN(1),SC(1),CVMA(1),CVAV(1),NTND(1),
1THT(1),TDEN(1),ST(1),TVMA(1),TVAV(1)
PRINT 107,NCRU(2),CHT(2),CDEN(2),SC(2),CVMA(2),CVAV(2),NTND(2),
1THT(2),TDEN(2),ST(2),TVMA(2),TVAV(2)
PRINT 107,NCRU(3),CHT(3),CDEN(3),SC(3),CVMA(3),CVAV(3),NTND(3),
1THT(3),TDEN(3),ST(3),TVMA(3),TVAV(3)
PRINT 200,CDA,SIGC,CVMAA,CVAVA,TDA,SIGT,TVMAA,TVAVA
PRINT 203,BC
PRINT 201,CU
PRINT 202, PHI
K=K+1
IF (N1+N2-K) 8,8,4
8 CALL EXIT
100 FORMAT (3I5,F5.2,F5.0,F6.3)F3.0,3I3)
101 FORMAT (3I5,F5.2,F5.0,F6.3)F3.0)

```

כ
ז
ח

PAULE
3

Filmed as received
without Page 3.

UNIVERSITY MICROFILMS.

0048 00814 0108 06300 0120 06426 0004 00922 0102 05660 0009 01274 0000 01050 0100 05484
 0050 01490 0109 06328 0101 05362 0020 03224 0103 05710 0106 05972 0107 06194 0200 06494
 0203 06780 0201 06604 0202 06684 0008 05470

06890 CORES USED
 35959 NEXT COMBON
 END OF COMPILEATION

MAIN 14300 06890 LOADED
 15 21190 00538 LOADED
 14 21728 01258 LOADED

ASPHALT CONTENT 4.5 P.C.
ASBESTOS CONTENT 0.0 P.C.

COMPRESSION

TENSION

SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS
4510	2.70	2.300	59.4	16.5	10.5	4510	2.55	2.0	16.2
4520	2.70	2.295	50.6	16.7	10.7	4520	2.58	1.5	17.0
4530	2.70	2.298	47.3	16.6	10.5	4530	2.64	1.8	16.9
AVERAGES		2.297	52.4	16.6	10.6		2.295	1.8	16.7

BEARING CAPACITY = 1500. P.S.I.

COHESION 4.8 P.S.I.

ANGLE PHI 65.0 DEGREES

ASPHALT CONTENT 5.0 P.C.
ASBESTOS CONTENT 0.0 P.C.

COMPRESSION

TENSION

SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS
5010	2.70	2.315	51.6	16.5	9.3	5010	2.58	2.7	16.8
5020	2.70	2.310	45.9	16.6	9.4	5020	2.56	1.7	17.1
5030	2.70	2.305	44.2	16.8	9.7	5030	2.60	0.0	17.7
AVERAGES		2.306	47.2	16.6	9.5		2.295	2.2	17.2

BEARING CAPACITY = 1041. P.S.I.

COHESION 5.1 P.S.I.

ANGLE PHI 65.3 DEGREES

ASPHALT CONTENT 5.5 P.C.
ASBESTOS CONTENT 0.0 P.C.

COMPRESSION

TENSION

SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS
5510	2.70	2.325	46.0	16.4	8.2	5510	2.50	0.0	16.1
5520	2.70	2.326	47.0	16.4	8.1	5520	2.50	2.4	16.3
5530	2.70	2.339	50.2	15.9	7.6	5530	2.50	2.8	16.2
AVERAGES		2.330	47.7	16.3	8.0		2.531	2.6	16.2

BEARING CAPACITY = 913. P.S.I.

COHESION 5.6 P.S.I.

ANGLE PHI 63.5 DEGREES

ASPHALT CONTENT 6.0 P.C.
ASBESTOS CONTENT 0.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
6010	2.70	2.361	56.2	15.5	6.1		6010	2.50	2.341	0.0	16.3	6.5	
6020	2.70	2.344	49.7	16.2	6.8		6020	2.50	2.359	4.0	15.6	6.2	
6030	2.70	2.357	50.2	15.7	6.3		6030	2.50	2.351	3.2	15.9	6.5	
AVERAGES		2.354	52.1	15.8	6.4			2.350		3.6	15.9	6.5	

BEARING CAPACITY = 809. P.S.I.
 COHESION 6.8 P.S.I.
 ANGLE PHI 60.5 DEGREES

ASPHALT CONTENT 6.5 P.C.
 ASBESTOS CONTENT 0.0 P.C.

COMPRESSION						TENSION					
SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR Voids	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR Voids
6510	2.50	2.346	38.8	16.2	6.1	6510	2.25	2.341	2.2	16.7	6.3
6520	2.50	2.351	38.6	16.3	5.9	6520	2.49	2.361	2.5	15.9	5.5
6530	2.50	2.361	47.0	15.9	5.5	6530	2.55	2.355	0.0	16.2	5.7
AVERAGES		2.352	41.4	16.2	5.8			2.352	2.5	16.3	5.8

BEARING CAPACITY = 719. P.S.I.
 COHESION 5.1 P.S.I.
 ANGLE PHI 62.2 DEGREES

ASPHALT CONTENT 7.0 P.C.
 ASBESTOS CONTENT 0.0 P.C.

COMPRESSION						TENSION					
SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS
7040	2.51	2.376	53.8	15.8	4.2	7040	2.49	2.369	3.8	16.1	4.5
7050	2.55	2.372	59.7	15.9	4.4	7050	2.52	2.350	4.1	15.7	5.3
7060	2.50	2.367	50.2	16.1	4.6	7060	2.52	2.360	3.8	16.4	4.8
AVERAGES		2.371	54.5	16.0	4.4			2.359	3.9	16.4	4.9

BEARING CAPACITY = 812. P.S.I.
 COHESION 7.3 P.S.I.
 ANGLE PHI 59.9 DEGREES

ASPHALT CONTENT 8.0 P.C.
 ASBESTOS CONTENT 0.0 P.C.

COMPRESSION							TENSION						
SAMPLE		HEIGHT	DENSITY	STRESS	VMA	AIR Voids	SAMPLE		HEIGHT	DENSITY	STRESS	VMA	AIR Voids
8010		2.42	2.372	48.2	16.7	3.1	8010		2.50	2.371	5.1	16.8	3.2
8020		2.46	2.376	50.6	16.6	3.0	8020		2.49	2.356	5.7	17.3	3.8

8030 2.46 2.377 52.2 16.6 2.9 8030 2.50 2.364 5.4 17.0 3.4
 AVERAGES 2.375 50.4 16.6 3.0 2.363 5.4 17.0 3.5

BEARING CAPACITY = 516. P.S.I.
 COHESION 8.2 P.S.I.
 ANGLE PHI 53.6 DEGREES

ASPHALT CONTENT 5.0 P.C.
 ASBESTOS CONTENT 2.0 P.C.

COMPRESSION			TENSION		
SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS
5012 2.54	2.366	67.5 14.6 7.4	5012 2.53	2.320	6.3 16.2 9.2
5022 2.55	2.326	71.7 16.0 8.9	5022 2.53	2.307	8.8 16.7 9.7
5032 2.52	2.334	83.8 15.7 8.6	5032 2.55	2.290	5.1 17.3 10.3
AVERAGES	2.342	74.3 15.4 8.3		2.302	5.7 16.7 9.7

BEARING CAPACITY = 1030. P.S.I.
 COHESION 10.3 P.S.I.
 ANGLE PHI 58.8 DEGREES

ASPHALT CONTENT 6.0 P.C.
 ASBESTOS CONTENT 2.0 P.C.

COMPRESSION			TENSION		
SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS
6012 2.50	2.381	86.3 14.6 5.5	6012 2.50	2.397	6.3 14.3 4.8
6022 2.49	2.373	82.8 15.1 5.8	6022 2.53	2.320	6.1 17.0 7.9
6032 2.52	2.362	78.5 15.5 6.2	6032 2.55	2.335	7.0 16.5 7.3
AVERAGES	2.372	83.2 15.2 5.6		2.350	7.1 15.9 6.7

BEARING CAPACITY = 1047. P.S.I.
 COHESION 12.2 P.S.I.
 ANGLE PHI 57.2 DEGREES

ASPHALT CONTENT 7.0 P.C.
 ASBESTOS CONTENT 2.0 P.C.

COMPRESSION			TENSION		
SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS VMA AIR VOIDS
7012 2.50	2.394	85.0 15.2 3.6	7012 2.47	2.394	11.0 15.2 3.6
7022 2.53	2.398	85.5 15.0 3.5	7022 2.54	2.373	9.2 15.9 4.5
7032 2.49	2.403	90.0 14.8 3.3	7032 2.49	2.397	11.2 15.1 3.5
AVERAGES	2.398	86.8 15.0 3.5		2.388	10.5 15.4 3.9

BEARING CAPACITY = 804. P.S.I.

COHESION 15.1 P.S.I.
ANGLE PHI 51.6 DEGREES

ASPHALT CONTENT 8.0 P.C.
ASBESTOS CONTENT 2.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
8012	2.40	2.398	76.4	15.6	2.2	2.2	8012	2.50	2.399	13.7	15.8	2.1	
8022	2.50	2.397	72.5	15.6	2.2	2.2	8022	2.51	2.397	14.9	15.8	2.2	
8032	2.50	2.402	79.0	15.7	2.0	2.0	8032	2.51	2.400	14.9	15.7	2.1	
AVERAGES		2.399	76.0	15.6	2.1	2.1		2.398	14.3	15.8	2.2		

BEARING CAPACITY = 472. P.S.I.

COHESION 16.6 P.S.I.
ANGLE PHI 42.7 DEGREES

ASPHALT CONTENT 5.0 P.C.
ASBESTOS CONTENT 4.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
5014	2.59	2.373	95.6	14.3	7.2	7.2	5014	2.53	2.357	13.4	14.9	7.8	
5024	2.50	2.333	96.9	15.6	8.8	8.8	5024	2.58	2.319	9.6	16.3	9.3	
5034	2.62	2.315	81.1	16.4	9.5	9.5	5034	2.57	2.324	10.1	16.1	9.1	
AVERAGES		2.340	91.2	15.5	8.5	8.5		2.333	11.0	15.7	8.8		

BEARING CAPACITY = 844. P.S.I.

COHESION 15.8 P.S.I.
ANGLE PHI 51.6 DEGREES

ASPHALT CONTENT 6.0 P.C.
ASBESTOS CONTENT 4.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
6014	2.56	2.375	103.0	15.0	5.8	5.8	6014	2.55	2.349	12.3	16.0	6.9	
6024	2.58	2.365	93.5	15.4	6.2	6.2	6024	2.52	2.365	12.3	15.4	6.2	
6034	2.57	2.367	99.2	15.3	6.1	6.1	6034	2.57	2.351	10.1	15.9	6.8	
AVERAGES		2.369	98.6	15.3	6.1	6.1		2.355	11.6	15.8	6.6		

BEARING CAPACITY = 935. P.S.I.

COHESION 16.9 P.S.I.
ANGLE PHI 52.0 DEGREES

ASPHALT CONTENT 7.0 P.C.
ASBESTOS CONTENT 4.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
7014	2.51	2.406	105.9	14.7	3.3		7014	2.55	2.389	13.0	15.3	4.0	
7024	2.57	2.400	96.7	15.0	3.5		7024	2.55	2.389	13.3	15.3	4.0	
7034	2.55	2.404	104.2	14.8	3.4		7034	2.55	2.389	14.2	15.3	4.0	
AVERAGES		2.403	102.3	14.8	3.4			2.389	14.2	15.3	4.0		

BEARING CAPACITY = 839. P.S.I.
COHESION 19.0 P.S.I.
ANGLE PHI 49.1 DEGREES

ASPHALT CONTENT 8.0 P.C.
ASBESTOS CONTENT 4.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
8014	2.54	2.402	79.5	15.7	2.2		8014	2.53	2.398	16.1	15.8	2.3	
8024	2.54	2.397	62.6	15.0	2.4		8024	2.50	2.404	16.4	15.6	2.1	
8034	2.44	2.388	62.2	16.2	2.7		8034	2.49	2.402	17.0	15.7	2.2	
AVERAGES		2.395	81.5	15.9	2.4			2.401	16.5	15.7	2.2		

BEARING CAPACITY = 483. P.S.I.
COHESION 10.3 P.S.I.
ANGLE PHI 41.4 DEGREES

ASPHALT CONTENT 5.0 P.C.
ASBESTOS CONTENT 6.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
5016	2.64	2.327	92.6	16.0	9.1		5016	2.63	2.302	11.4	16.9	10.1	
5026	2.63	2.296	79.2	17.1	10.4		5026	2.57	2.315	11.5	16.4	9.6	
5036	2.63	2.302	81.2	16.9	10.1		5036	2.60	2.307	10.9	16.7	9.5	
AVERAGES		2.308	84.3	16.6	9.9			2.308	11.2	16.7	9.9		

BEARING CAPACITY = 718. P.S.I.
COHESION 15.3 P.S.I.
ANGLE PHI 49.9 DEGREES

ASPHALT CONTENT 6.0 P.C.
ASBESTOS CONTENT 6.0 P.C.

COMPRESSION

TENSION

SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
6016	2.58	2.347	94.9	16.0	7.1		6016	2.50	2.366	16.6	15.4	6.3	
6026	2.50	2.323	86.7	16.9	8.0		6026	2.54	2.347	16.0	16.0	7.1	
6036	2.48	2.372	107.9	15.2	6.1		6036	2.53	2.362	15.8	15.5	6.5	
AVERAGES		2.347	96.5	16.0	7.0				2.358	16.1	15.6	6.6	

BEARING CAPACITY = 873. P.S.I.
 COHESION 19.7 P.S.I.
 ANGLE PHI 45.4 DEGREES

ASPHALT CONTENT 7.0 P.C.
 ASBESTOS CONTENT 6.0 P.C.

COMPRESSION							TENSION						
ABSOLUTE COMPRESSION							ABSOLUTE TENSION						
SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
7016	2.57	2.403	106.3	14.8	5.5		7016	2.54	2.403	18.8	14.8	5.5	
7026	2.57	2.401	115.5	14.9	5.6		7026	2.57	2.396	18.2	15.0	5.7	
7036	2.54	2.394	109.9	15.2	5.9		7036	2.57	2.402	18.6	14.9	5.6	
AVERAGES									2.401	18.5	14.9	5.6	

BEARING CAPACITY = 777. P.S.I.
 COHESION 22.7 P.S.I.
 ANGLE PHI 45.5 DEGREES

ASPHALT CONTENT 6.0 P.C.
 ASBESTOS CONTENT 6.0 P.C.

COMPRESSION							TENSION						
ASBESTOS CONTENT 0.01%													
SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
8016	2.55	2.385	100.9	16.3	3.0		8016	2.52	2.386	17.7	16.2	2.5	
8026	2.52	2.382	93.1	16.4	3.1		8026	2.54	2.393	18.5	16.0	2.7	
8036	2.55	2.368	94.8	16.9	3.7		8036	2.55	2.376	18.0	16.6	3.3	
AVERAGES		2.378	96.3	16.5	3.2				2.385	18.1	16.3	3.0	

BEARING CAPACITY = 607. P.S.I.
 COHESION 20.8 P.S.I.
 ANGLE PHI 43.0 DEGREES

ASPHALT CONTENT 6.0 P.C.
 ASBESTOS CONTENT 6.0 P.C.

COMPRESSION							TENSION						
ASBESTOS CONTENT 0.0 P.C.													
SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS	SAMPLE	HEIGHT	DENSITY	STRESS	VMA	AIR	VOIDS
6040	2.44	2.354	166.8	15.8	6.4		6040	2.50	2.335	24.1	16.5	7.1	

6050	2.55	2.336	148.3	16.4	7.1	6050	2.52	2.326	21.7	16.8	7.5
6060	2.47	2.341	156.4	16.3	6.9	6060	2.52	2.317	20.3	17.1	7.9
AVERAGES	2.543	157.2	16.2	6.8			2.526		22.0	16.8	7.5

BEARING CAPACITY = 1277. P.S.I.
 COHESION 29.4 P.S.I.
 ANGLE PHI 48.9 DEGREES

ASPHALT CONTENT 7.0 P.C.
 ASBESTOS CONTENT 0.0 P.C.

COMPRESSION						TENSION					
SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS		
7070	2.53	2.381	155.2	15.6	4.0	7010	2.51	2.362	21.8	16.3	4.8
7080	2.50	2.374	145.6	15.9	4.3	7020	2.49	2.356	24.8	16.5	5.0
7090	2.53	2.372	139.7	15.9	4.4	7030	2.50	2.352	23.6	16.7	5.2
AVERAGES	2.379	146.8	15.8	4.2			2.556		23.4	16.5	5.0

BEARING CAPACITY = 1060. P.S.I.
 COHESION 29.3 P.S.I.
 ANGLE PHI 46.4 DEGREES

ASPHALT CONTENT 6.0 P.C.
 ASBESTOS CONTENT 4.0 P.C.

COMPRESSION						TENSION					
SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS		
6044	2.58	2.371	231.8	15.2	6.0	6044	2.50	2.378	45.9	14.9	5.7
6054	2.58	2.350	196.6	15.9	6.8	6054	2.56	2.359	39.2	15.6	6.5
6064	2.57	2.360	221.4	15.6	6.4	6064	2.57	2.346	35.1	16.1	7.0
AVERAGES	2.360	216.6	15.6	6.4			2.561		40.1	15.5	6.4

BEARING CAPACITY = 1380. P.S.I.
 COHESION 46.6 P.S.I.
 ANGLE PHI 43.4 DEGREES

ASPHALT CONTENT 7.0 P.C.
 ASBESTOS CONTENT 4.0 P.C.

COMPRESSION						TENSION					
SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS	SAMPLE HEIGHT	DENSITY	STRESS	VMA	AIR VOIDS		
7044	2.55	2.396	218.3	15.1	3.7	7044	2.53	2.407	49.3	14.7	3.2
7054	2.56	2.393	209.7	15.2	3.8	7054	2.56	2.403	47.5	14.8	3.4
7064	2.56	2.402	235.4	14.9	3.4	7064	2.54	2.400	47.2	15.0	3.5
AVERAGES	2.397	221.1	15.1	3.6			2.403		48.0	14.8	3.4

BEARING CAPACITY = 1239. P.S.I.
COHESION 51.2 P.S.I.
ANGLE PHI 40.0 DEGREES