STRENGTH AND VOLUME CHANGE
CHARACTERISTICS OF BITUMINOUS MIXTURES

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PURDUE UNIVERSITY
LAFAYETTE INDIANA

by

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Technical Paper

STRENGTH AND VOLUME CHANGE CHARACTERISTICS
OF BITUMINOUS MIXTURES

TO: K. B. Woods, Director
Joint Highway Research Project

November 2, 1960

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 2-4-16
Project: C-36-6P

Attached is a technical paper entitled "Strength and Volume Characteristics of Bituminous Mixtures" which has been authored by Messrs. J. H. Schaub and W. H. Goetz of our staff. This paper has been prepared for presentation to the Annual Meeting of the Highway Research Board in Washington, D. C. in January 1961.

The paper contains a summary of the research performed by Mr. Schaub for his Ph.D. degree. The material contained in the report was presented to the Board at a previous meeting.

The paper is presented to the Board for the record and for approval of the presentation as indicated.

Respectfully submitted,

Harold L. Michael

Harold L. Michael, Secretary

HLM:kmc

Attachment

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Technical Paper

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OF BITUMINOUS MIXTURES

by

J. H. Schaub*

and

W. H. Goetz**

Prepared for presentation to
Annual Meeting, Highway Research Board
Washington, D. C.

January, 1961

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Project No: C-36-6P

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INTRODUCTION

For approximately twenty-five years the attention of paving design engineers has been directed in part towards the use of triaxial shear strength methods of test to evaluate the properties of bituminous mixtures. Interest in this form of test, particularly as a means of evaluating fundamental strength properties of bituminous mixtures, has become increasingly great. At the present time there are several techniques in use that utilize the principles of triaxial shear testing for the design and strength evaluation of bituminous concrete mixtures.

Present practice in the triaxial testing of bituminous mixtures does not consider the effect of the method of performance of the test on observed shear strength values. Though air voids are of major interest in bituminous mixture design, little or no attention has been devoted to the evaluation of the void content of bituminous mixtures at failure criteria.

It appeared desirable to establish whether the method of performance of a triaxial test on bituminous mixtures affects the observed shear strength of the mixtures. In addition, a knowledge of the magnitude of change, if any, in the void content of a mixture during shear would aid in establishing a proper perspective of the role of air voids in the design of an adequate bituminous mixture for road use.

The Specific purposes of the study were as follows:

1. To Determine the effect of the method of performing a triaxial test on the observed shear strength values and to evaluate the void content of a specimen at any point during the progress of a test.
2. To establish whether there is a change in the volume of a bituminous mixture subjected to a triaxial stress system, and, if so, to establish the significant change in voids that is reflected by a change in observed shear strength.
3. To investigate whether there is a unique relationship between the void content and the shear strength at any given failure criterion for a bituminous mixture prepared to a given set of initial conditions.

MATERIALS AND PROCEDURES

Materials

The bituminous concrete mixture utilized for this study was chosen to be typical of such mixtures in use at the present time by various agencies. It was not intended that the mixture satisfy the specifications of any particular organization. However, the mixture does satisfy, in general, the requirements of the State Highway Department of Indiana for Hot Asphaltic Concrete Surface, Type A (1).

The composition of the aggregate blend is shown in Figure 1 together with the gradation curves of two other mixtures presented for comparison purposes.

The coarse aggregate fraction (percent retained on No. 4 sieve) of the aggregate blend was a crushed and washed limestone characterized by a low porosity and a fine-grained texture. The aggregate passing the No. 4 sieve was composed of three materials. A river terrace sand was used for the fraction No. 4 - No. 100 and a very uniform dune sand formed the portion between the No. 100 - No. 200 sieves. A Type 1 Portland cement was utilized for the filler material.

The specific gravity and absorption values for the aggregates are shown in Table 1. The cement was found to have a specific gravity of 3.15.

The absorption of bitumen by the aggregate blend was established by the Maximum Theoretical Density (Rice) (2) procedure to be 0.72 percent.

Preparation of Specimens

Aggregates were batched cold to the chosen aggregate gradation formula. Blended aggregates and asphalt were heated separately to $300^{\circ}\text{F} \pm 10^{\circ}$ and, after reaching this temperature, combined in the desired amounts. Mixing was performed with an electric mixer to a period of two minutes.

FIGURE 1 . AGGREGATE GRADATION FOR TEST MIXTURE

TABLE 1

Specific Gravity and Absorption of Aggregates

	<u>Limestone</u>	<u>River Sand</u>	<u>Dune Sand</u>
Bulk	2.660	2.574	2.595
Apparent Specific Gravity	2.700	2.715	2.656
% Water Absorption	0.63	2.01	0.88

The asphalt used for the study was an 85-100 penetration grade provided by the Texas Company. The test characteristics of the asphalt are shown in Table 2.

TABLE 2

Results of Tests on Asphalt Cement

Penetration, 100 g., 5 sec., 77°F	93
Penetration, 100 g., 5 sec., 32°F	21
Softening Point, Ring and Ball, °F	116
Ductility, 77°F, cm.	200+
Flash Point, Cleveland Open Cup, °F	590
Solubility in CCl ₄ , %	99.86
Spot Test	Negative
Specific Gravity, 77/77 °F	1.033

The aggregate blend chosen and the range of asphalt contents considered for the testing program were evaluated for general suitability as an asphalt paving mixture by means of the Marshall (3) and Stabilometer test procedures (4). The results of these characterization tests are shown in Table 3.

TABLE 3

Results of Characterization Tests

Marshall Method Results

<u>Asphalt Content % by weight of mix</u>	<u>Stability lbs.</u>	<u>Flow 1/100 in.</u>	<u>Unit Weight lb/cu. ft.</u>	<u>Voids %</u>	<u>Voids Filled %</u>
3.75	1105	11.5	145.7	6.2	58.0
4.25	1165	13.3	147.2	4.6	69.0
4.85	1125	13.8	149.1	2.6	82.0
5.19	1232	14.3	149.5	1.8	87.5
5.74	1050	15.9	149.1	1.1	92.5

Stabilometer Method Results

CKE, fine aggregate	2.2
Oil Equivalent, coarse aggregate	3.3
Surface Area, sq.ft/lb.	18.5
Optimum Asphalt Content, % by weight of aggregate	5.2

<u>Asphalt Content % by weight of mix</u>	<u>by weight of aggregate</u>	<u>Stabilometer Value</u>	<u>Unit Weight lb/cu. ft.</u>	<u>Voids %</u>	<u>Voids Filled %</u>
4.06	4.23	45	150.4	4.7	67.5
4.27	4.46	43	151.4	3.9	72.7
4.77	5.01	37	152.7	2.4	83.0
5.32	5.63	31	152.3	1.7	88.5

Two mix batches were required for one specimen. While the second batch was being prepared, the first was stored in a shallow, flat-bottomed metal pan at a temperature of 140°F. The two batches were combined by hand mixing and the total aggregate and asphalt weights were used to compute the asphalt content by weight of mixture.

Prior to compaction the material for each specimen was cured in a shallow pan at 140°F in a forced-draft oven for a period of 18 - 20 hours.

Compaction

Kneading compaction seemed to offer the best possibility for providing a desirable specimen and was consonant with the present trend of thought regarding the formation of a specimen most nearly like the pavement prototype.

The principles of kneading compaction have been discussed frequently as has been the operation of the kneading compactor (5) (6) (7). In general, the compaction process is accomplished as follows: Material is fed into the mold which is mounted on a rotating table; the compaction foot, a pie-shaped segment with an area approximately one-fourth of that of the mold, is activated by a combination mechanical and hydraulic-pneumatic control system. The foot moves downward against the sample until a pre-set load is built up. The load is maintained for a short interval of time and then released. A load cycle requires two seconds to complete. Means are available for adjusting the foot pressure over a wide range. Material may be placed in the mold in layers or by a continuous flow process during compaction.

The kneading compactor model available for use is shown in Figure 2. It was constructed by the August Manufacturing Company of Oakland, California. A special mold, Figure 3, suitable for the formation of specimens eight inches high

FIGURE 2. KNEADING COMPACTOR

FIGURE 3. MOLD FOR TRIAXIAL TEST SPECIMENS

by four inches in diameter was constructed in the shops of the School of Civil Engineering.

The criteria for a suitable specimen were considered to be uniform density and asphalt content throughout the height of the sample. Uniformity of these two measures implies a uniform distribution of voids throughout the specimen. Void uniformity was considered desirable in light of the purposes of the study. To evaluate the uniformity of asphalt content and density, each specimen made for purposes of developing a compaction procedure was cut into three sections perpendicular to the axis of the sample. Each section was measured, weighed, and the volume computed by displacement in water using the principle of Archimedes and by computation using the specimen dimensions. The sections were then treated for the extraction of asphalt using ASTM Method D - 1097 (8).

Absolute uniformity of density or asphalt content was considered to be unattainable from a practical standpoint. It was decided, therefore, to strive for a procedure that would yield density values of the sections within one percent of the average of the three sections and asphalt contents of the sections within three-tenths of one percent of the average asphalt content of the three sections as established from extraction test results.

Numerous trial compaction procedures were investigated. The procedure finally settled upon was as follows:

1. The mixture was removed from the 140°F curing oven after 18 - 20 hours of curing and was remixed by hand.
2. The mixture was split into two approximately equal parts and each part brought to a temperature of 230°F $\pm$ 10°.
3. One-quarter of the mixture was introduced into the preheated mold which was placed on the rotating table of the kneading compactor.

4. To reduce the tendency for surface voids to develop in the lower portion of the sample, the first one-quarter of the mixture was rodded with a 3/4 inch bullet-nose rod. Twenty tamps were applied to the center of the mix and twenty around the periphery of the mix.
5. The material in the mold was subjected to 15 tamps of the tamping foot under the desired and pre-set foot pressure.
6. The remainder of the sample was introduced into the mold while the compactor was in operation. One-third of this portion (one-quarter of the total sample) was added gradually over each of three two-minute periods. All material was stored in the heating oven at a temperature of $230^{\circ}\text{F} \pm 10^{\circ}$ until ready for compaction.

This procedure permitted the application of 180 tamps on three-fourths of the original sample while this quantity was being added to the mold.
7. Two minutes of tamping (60 tamps) was applied to the surface of the entire sample.
8. The mold was removed from the kneading compactor and the sample subjected to a double plunger static load of 12600 pounds as a leveling load. This load was applied at a rate of 0.025 inches of deformation per minute and was released immediately upon reaching the load specified. the loading was applied by the use of a Riehle Testing Machine, 50,000 pound capacity, and having a variable speed drive.
9. The mold was allowed to cool in air for approximately 30 minutes, at which time the mold was disassembled and the specimen removed.
10. Each specimen was marked for identification. For purposes of establishing the uniformity of the specimens, the bulk density of each was determined from the dimensions of the specimen.

11. All specimens were stored in a constant temperature atmosphere of 77°F until time for testing.

In all cases the uniformity of density was well within the limit sought and generally the density of the cut sections varied from the average by less than one-half of one percent. The uniformity of asphalt content that was desired was attained in all specimens.

Controlled Temperature Room

In order to eliminate the variable of temperature and its effects upon the tests performed in this study, a controlled-temperature atmosphere was considered necessary. A commercially available unit marketed by The Electric Hotpack Company, Inc., Philadelphia, Pennsylvania, under the name Controlled Enviromental Room, Model 8316, was found to satisfy all requirements of the project and was obtained for use.

Figure 4 is a photograph of the exterior of the controlled-temperature room while Figure 5 shows the interior of the room with all testing equipment in place. Figure 6 shows a close-up view of the control panel and recording unit on the exterior of the room.

The recording unit provided a continuous record of the room's temperature through a 24 hour period. Tests conducted during the early period of operation of the room indicated that the recorded trace was an accurate measure of the dry-bulb temperature of the room at the sensing element. A series of accurate thermometers were placed at points of interest throughout the room and observed carefully for long periods to note the variation of temperature within the room. Average air temperatures at the triaxial cell and at the volume measuring device were observed to be 0.3°C above the temperature at the sensing element. Temperature readings taken every minute on all thermometers indicated

**FIGURE 4. CONTROLLED TEMPERATURE ROOM-
EXTERIOR VIEW**

**FIGURE 5. CONTROLLED TEMPERATURE ROOM—
INTERIOR VIEW**

FIGURE 6. CONTROL PANEL, RECORDING UNIT AND TYPICAL TEMPERATURE RECORD FOR CONTROLLED TEMPERATURE ROOM

a cycling of air temperature from approximately 1°C above the average to approximately 1°C below the average with the cycling occurring approximately once every ten minutes.

Observations on the water temperature within the elements of the volume measuring device showed a maximum variation of $\pm 0.1^{\circ}\text{C}$ from the temperature at the sensing element. In summary, all temperature observations indicated that the average temperature at any point within the room was within the desired maximum temperature range of $\pm 0.5^{\circ}\text{C}$. A typical temperature record, as made by the Partlow recorder, is shown in Figure 6.

Volume Measuring Device

In order to follow the change of void content during the progress of a triaxial test, equipment was constructed that provided a measure of the change of volume of the specimen during a test. The equipment developed was adopted from that utilized at the Soil Mechanics Laboratory at the University of London and reported by Bishop and Henkel (9). Figure 7 is a photograph of the device used in this study while Figure 8 is a diagrammatic sketch of the volume measuring device that also shows its relationship to the triaxial cell and the pressure source. The key following these figures identifies the principal parts of the apparatus (Table 4).

The basic principle upon which the volume measuring device was operated is simple. Any change in the volume of the sample was reflected by a change in the volume of the liquid in the triaxial cell. The magnitude of this volume change was measured by a change in the level of the mercury column in the measuring tube. A differential head of mercury between the measuring tube and the mercury reservoir tube was prevented by mounting the latter tube on a

FIGURE 7. VOLUME MEASURING DEVICE

FIGURE 8. DIAGRAMMATIC SKETCH OF VOLUME MEASURING DEVICE

TABLE 4

Key to Details of Figure 8

1. Nitrogen tank
2. Pressure regulator
3. Quick release pressure coupling
4. Water reservoir tank
5. Water reservoir tank outlet valve
6. Mercury reservoir control valve
7. Polyethylene plastic tubing (1/4 inch O.D.)
8. Calibrated spring
9. Plastic mercury reservoir tube
10. Plastic measuring tube
11. Metric scale
12. Polyethylene plastic tubing (1/4 inch O.D.)
13. Bypass valve
14. Measuring tube control valve
15. Triaxial cell water-inlet valve
16. Triaxial cell
17. Triaxial cell air-escape valve
18. Sample drainage valve

calibrated spring that changed length as the amount of mercury in the reservoir tube changed.

The volume measuring device was operated as follows: The triaxial cell was filled with the confining liquid, water in this case, with care taken to remove all air bubbles from the cell. At this point, the water in the cell was continuous to the water in the reservoir while the flexible tubing and the measuring tubes were completely filled with water and mercury. Valves were adjusted so that a pressure applied to the water reservoir was transmitted through the flexible tubing to the liquid in the cell. Upon the application of pressure to the cell liquid (the confining pressure of the test) a change in the level of the mercury in the measuring tube took place. This change reflected the increase in the volume of the cell liquid that resulted from the lateral pressure forcing the impervious membrane into the surface voids of the specimen. The change in volume of the specimen with time under any load application to the specimen caused a change in the level of the mercury in the measuring tube. For example, if the specimen decreased in volume under load, water entered the cell from the flexible tubing and the mercury level of the measuring tube moved upwards an amount equal to an equivalent volume. Simultaneously the mercury level in the reservoir tube dropped. The decrease in weight of the reservoir tube caused the calibrated spring to shorten until a uniform level of mercury existed between the two mercury tubes.

The metric scale located adjacent to the measurement tube permitted a reading of the change in level of the mercury to the nearest one millimeter which corresponded to a volume change of 0.497 cubic centimeters. A volume change of this magnitude is equivalent to a change in volume of the specimen of approximately 0.03 percent. It should be noted that any observed change in mercury level must be corrected for any variation in temperature, for the change

in volume of the cell under the applied confining pressure, and for the volume displaced by the movement of the piston.

Temperature changes were minimized by the use of the controlled-temperature atmosphere in which all tests were performed. A maximum temperature variation of one-half degree Centigrade was established for the test location. With these temperature conditions the maximum possible variation of the volume of the cell liquid and the volume of the cell has been computed to be ± 0.255 cubic centimeters or a variation in mercury level of approximately one-half millimeter.

The change in the volume of the cell and the volume measuring device under the applied confining pressure was established experimentally by calibration tests. A steel dummy specimen the same size as a bituminous specimen was used to simulate it and the cell was assembled as for a routine test. The changes in volume of the apparatus under the various confining pressures to be used were observed and recorded as calibration factors for the assembly. The change in volume of the assembly under a confining pressure occurred almost instantaneously upon the application of the pressure. Little change in volume of the apparatus occurred with time under prolonged application of the confining pressure.

The entrance of the piston under load into the triaxial cell caused a continuing displacement of the cell liquid that resulted in a decrease in the level of the mercury in the measuring tube. Computations based on the diameter of the piston and the deformation of the specimen permitted the evaluation of a correction factor which was applied to apparent volume changes.

Triaxial Testing Equipment

Figure 5 and Figure 9 are general views of the triaxial testing equipment used in this study. The former figure shows a front view of the testing frame in position within the controlled-temperature room while the latter figure is a

FIGURE 9. TRIAXIAL CELL IN LOADING FRAME

close-up of the loading crossbar of the frame with the triaxial cell in position. Figure 10 shows a section of the triaxial cell and should be used in conjunction with Table 5 which serves as a key to the major elements of the cell.

The triaxial cell was modified from one available in the Bituminous Mixtures Laboratory from previous work done by Oppenlander (10). The top and base of the cell were of aluminium and the cylinder was brass. Compression loads were applied to a loading cap resting in position on the specimen by means of a hardened steel piston. The loading cap was recessed to receive the spherical end of the piston. The piston was centered through a vertical ball bushing and grease seal in the center of the cell. Preliminary tests showed the grease seal unable to hold the higher lateral pressures without appreciable leaking. Consequently, an O-Ring seal was installed above the grease seal. Adjustment of thumb screws permitted a cover plate to be forced against the O-Ring which expanded against the piston to provide a water-tight seal. The remainder of the cell details are apparent upon reference to Figure 10 and to the identification key shown in Table 5.

The addition of the O-Ring seal was successful in eliminating leakage but it was apparent that this procedure materially increased frictional resistance to the piston. In order to evaluate piston friction, a proving ring was placed inside the cell and the cell was assembled using a clear lucite cylinder. The use of known loads on the hanger permitted a computation of the load applied to the piston. Evaluation of the load reaching the proving ring was possible through the calibration charts of the proving ring. The difference between these loads was assumed to be friction. A plot of load applied versus friction was developed and used to correct all applied loads in the computation of test results.

FIGURE 10. DETAILS OF TRIAXIAL CELL

TABLE 5

Key to Details of Figure 10

- 1. Dial gage, 0.01 mm.
- 2. Loading head support
- 3. Loading head
- 4. Piston
- 5. O-Ring seal assembly
- 6. Ball-bushing seal assembly
- 7. Air escape valve
- 8. Pressure relief valve
- 9. Cylindrical wall of triaxial cell
- 10. Upper head assembly
- 11. Tie rods
- 12. Upper loading cap
- 13. Upper drainage connection (sealed)
- 14. Rubber membrane with O-Ring seals
- 15. Specimen
- 16. Bottom loading cap
- 17. Bottom drainage connection with valve
- 18. Bottom head assembly
- 19. Water inlet connection

Triaxial Test Procedure

Preparation of Specimens for Testing

The preparation of a specimen for testing was a time-consuming operation requiring careful attention to details. The specimen to be tested was removed from storage in the controlled-temperature room and carefully centered on the bottom loading cap of the triaxial cell. The top loading cap was placed on the sample. The circumferential surfaces of both caps were covered with a thin film of Dcw-Corning Hi-Vacuum Silicone Grease. A thin rubber membrane, available from Soiltest, Inc., in a size adequate for four-inch diameter by eight-inch high specimens, was placed over the specimen utilizing a membrane stretcher for the operation. The membrane was smoothed carefully to the surface of the specimen and both loading caps. The membrane was sealed to the loading caps by the use of two O-Rings (2-7/8 inch diameter) at each end. A vacuum was applied to the specimen through the lower drainage connection. If the membrane held the vacuum it was assumed to be a leak-free set-up and the assembly of the cell continued. A photograph of a partially complete test assembly is shown in Figure 11.

This procedure was used for all samples except those to be tested with a 60 psi confining pressure. Experience indicated that one membrane was not capable, in general, of withstanding this level of confining pressure without rupturing or leaking. As a result, double membranes were used for all 60 psi confining pressure tests. The second membrane was placed over the first one and secured by an additional two O-Rings at each end. The double membrane procedure performed well for the high-confining-pressure tests.

The cell assembly was completed by putting the cell cylinder, the connecting rods, and the top including the piston into place. The connecting rods were fastened by tightening the rod nuts uniformly and the piston was pressed firmly

FIGURE 11. ASSEMBLY OF TRIAXIAL CELL

into place in the spherical seat of the upper loading cap. The assembled cell and specimen were placed in position within the triaxial frame.

Once the cell and specimen were placed in the loading frame the test progressed in a routine manner. The loading bar was brought into contact with the piston and adjusted to a balance in this position. The hanger loading arm was leveled and the deflection dial adjusted in place on the loading bar. The volume measuring device was attached to the water inlet and the cell was filled with water. During the filling operation, the valves to the measuring tube and mercury reservoir tube were closed. Gravity flow was not sufficient to fill the cell; therefore, a slight (5 to 8 psi) pressure was applied to the top of the reservoir tank. When water first flowed from the air-escape valve, the water-inlet valve and the water reservoir outlet valves were closed and the pressure connection to the water reservoir was disconnected. To check on the removal of all air from the cell, the water reservoir tank was manually elevated, the outlet valve opened, and the cell water-inlet valve opened until air-free water flowed from the air-escape valve. The two water valves were then closed and the reservoir tank returned to its support. With the cell full of water, the air-escape valve and the bypass valve were closed. The cell water-inlet valve and the valves to the mercury reservoir and the measuring tube were opened.

Loading Procedure for Routine Tests

With the completion of the above steps, the specimen was ready to test. An initial reading of the level of the mercury in the measuring tube and a deformation dial reading were obtained. The specimen drainage valve was adjusted in accordance to the type of test, closed for the "Quick" test, and open for all other types of tests. The desired confining pressure was applied to the water reservoir with adjustment of the pressure accomplished through a control valve on a tank of compressed nitrogen. To start the test, the water reservoir outlet

valve was opened permitting the test pressure to be applied to the system while simultaneously a weight was placed on the 10:1 load hanger to balance the cell pressure acting on the loading piston.

Regardless of the type of test being performed, readings of the volume measuring tube and deformation dial were obtained at 15 seconds after the application of the confining pressure. The 15-second reading was utilized during the computation stage as will be discussed under that heading.

In the performance of a "Quick" or "Q" test, a reading of both the volume measuring device and deformation dial was obtained at five minutes after the application of the confining pressure. At this time the first axial load was applied. Readings of the volume-measuring device and deformation dial were taken at 5-minute intervals just prior to the application of the next axial load increment. In the interval between applications of axial loads the leveling device was manipulated to maintain the hanger loading bar as close to horizontal as possible. Axial loads were applied in this fashion until failure of the sample occurred. Failure was considered to have taken place when (a) axial deformation exceeded one centimeter, (b) the leveling adjustment could not maintain pace with deformation and/or (c) the amount of volume change was so great as to present a danger of forcing the mercury level beyond the limits of the volume measuring device.

If "Consolidated Quick" or "Consolidated-Partially Drained" tests were being performed, readings were made of the volume measuring device and the deformation dial at times of 0.25, 0.5, 1, 2, 4, 8, 15, 30, and 60 minutes after the application of the confining pressure. A plot of the volume measuring device readings versus time (to a logarithmic scale) was prepared as the readings were observed. Full compaction of the specimen under the applied confining pressure

was assumed to have occurred if the 60-minute reading plotted as a straight line with the preceding points at a slope appreciably less than that defined by the plotted points at earlier times. This process is similar to that used for "time curves" for various load increments in consolidation tests of soils.

For "Consolidated Quick" or " Q_c " tests, the specimen drainage valve was closed at the 60-minute reading after the application of the lateral pressure. Axial load increments were applied and readings observed in the same way and at the same rate as for the previously described "Quick" tests. For "Consolidated-Partially Drained" or "CP" tests, the tests were completed in exactly the same fashion as for the " Q_c " tests except that the specimen drainage valve remained open throughout the test.

Tests at the three drainage conditions described were performed at confining pressures of 15, 30, and 60 psi on specimens made at 4 and 5 percent asphalt contents by weight of mixture and 150 and 400 psi compaction pressures. "Quick" tests at the same three confining pressures were performed on specimens prepared at 3, 4, and 5 percent asphalt contents by weight of mixture with compaction pressures of 250 and 500 psi. Additional "Quick" tests were performed on specimens fabricated at 150 and 400 psi. compaction pressures at an asphalt content of 3 percent by weight of mixture and using the specified confining pressures.

Slow Rate of Loading Tests

Three tests were performed at a confining pressure of 30 psi and using an interval of one hour between the application of axial load increments rather than the 5-minute interval used for the routine tests.

The specimens were prepared in the usual fashion at a single asphalt content and compaction foot pressure. One specimen was tested at each of the drainage

conditions represented by the "Q", "Q_c", and "CP" types of test. With the exception of the rate of axial load increment application, the tests were performed in exactly the same manner as the routine tests.

Water Saturated Tests

Two tests were performed after water saturation of specimens which had been compacted to a given set of initial conditions of asphalt content, compaction foot pressure, and air voids.

The water saturation was accomplished by placing the specimen in an air-tight chamber and subjecting it to a vacuum for a period of 30 minutes. Without releasing the vacuum, water was permitted to enter the chamber and cover the specimen. After an elapsed time of 30 minutes, the vacuum was applied again to the chamber and maintained for an additional half-hour. The application of the vacuum was discontinued and the specimen permitted to remain in the water within the evacuated atmosphere overnight or for a period of approximately 16 hours.

Upon removing the specimen from the chamber, it was measured for volume computations and weighed in a surface-dry condition to establish the amount of water that had been absorbed.

Testing was accomplished using a "Q" test at confining pressures of 15 and 60 psi. The cell was modified for these tests to insure no drainage by introducing solid aluminium loading caps at the top and bottom of the specimen. The solid loading caps replaced ones which contained grooves to facilitate drainage and which were provided with drainage outlets. The solid loading caps were used only for the water-saturated tests. In all other respects the testing procedure was similar to that previously described for the "Q" test.

Disassembly of Test Apparatus

At the completion of the test, the mercury reservoir and volume measuring tube valves were closed and the bypass valve opened. Pressure was released from the water reservoir tank, the cell air-escape valve was opened, and the cell liquid drained into the water reservoir tank. The deformation dial was moved clear of the loading crossbar and all load removed from the hangers. The cell water-inlet valve was closed before the cell was completely drained to prevent air from entering the plastic tubing system. The volume-measuring assembly was disconnected from the triaxial cell and the water remaining in the cell recovered in a glass cylinder for return to the water reservoir tank. The cell was removed from the controlled-temperature room for disassembly.

Upon removal of the brass cylinder from around the specimen, the specimen was inspected carefully for membrane leaks that may have occurred and which would have invalidated results. The membrane was stripped from the specimen and the cell cleaned thoroughly in preparation for the next test.

Evaluation of Test Procedure

Three problems were paramount in the triaxial testing procedure outlined above. The first was associated with the membranes. Occasional failures occurred and were apparent by leakage of liquid from the specimen drainage valve for tests where this valve was open, by continued unusual movements upward of the mercury level of the volume measuring tube, or by inspection of the membrane and specimen at the end of the test. In cases where such failures occurred the test results were disregarded and a new test performed. A second problem was leakage of the equipment during the test. Close observation of all seals, joints, etc. was maintained during all tests. If care was exercised in the set-up, no leaks occurred. The third problem involved the change in volume of the cell and the volume measuring equipment under

the application of the confining pressures. This problem was solved by periodic calibration tests made under all test confining pressures and by using a steel specimen. The volume changes observed during calibration trials took place almost instantaneously and were recorded for a correction of the 15-second readings taken during tests. As all other test data involved the use of differences in successive readings any errors due to equipment that may have been introduced in a particular test were contained within the 15-second reading.

RESULTS AND COMPUTATIONS

Observed Data

The compacted specimens were weighed to the nearest one-half gram and the diameter and height determined as an average of at least three measurements. All linear measurements were made to the nearest one-hundredth of a centimeter. These measurements provided the initial height and volume values of the specimen upon which all computations were based.

The weights of aggregate and asphalt recorded for each mixing operation permitted the computation of the average asphalt content by weight of mixture for each specimen. The measured weight, volume and average asphalt content of a specimen, together with the average bulk specific gravity of the aggregate blend, the specific gravity of the asphalt, and the percent absorption of asphalt by the aggregate made it possible to compute the volumes of aggregate, asphalt, and air in the specimen.

During the performance of each triaxial test, readings were taken at predetermined intervals of the extensometer used to measure the change in height of the specimen and of the level of the mercury in the volume measuring tube. These readings permitted the axial strain to be computed for each load increment and provided a means of establishing the change in volume of the specimen under the various loads.

A typical mix record, initial characteristics computation, and test data sheets and computations are shown in Tables 6, 7, 8, 9, and 10.

Stress-Strain Relationships

Loads were applied to the test specimen in multiples of a nominal 10 psi to a four-inch diameter specimen. A correction to the applied load was made for the

TABLE 6

Typical Mix Record

Specimen Q-12

	<u>Batch 1</u>	<u>Batch 2</u>
Tare Weight of Bowl, g	3141	3140
Weight of Bowl + Aggregate, g	5039	5038
Weight of Aggregate, g	1898	1898
Weight of Bowl + Aggregate + Asphalt, g	5139	5138
Weight of Asphalt, g	100	100
Weight of Mixture, g	1998	1998
Asphalt Content of Batch, %		
by weight of mixture	5.005	5.005
by weight of aggregate	5.269	5.269
Asphalt Content of Specimen, %		
by weight of mixture	5.005	
by weight of aggregate	5.269	
Specimen Diameter, cm	10.19, 10.19, 10.19 : Average = 10.19	
Specimen Weight, cm	19.95, 19.90, 19.85 : Average = 19.90	
Specimen Volume, cc	1622.85	
Specimen Weight, g	3977.0	
Specimen Density, g/cc	2.451	
Specimen Density, pcf	152.942	

TABLE 7

Typical Specimen Initial Characteristics Computations

Specimen Q-12

Asphalt Content, % by weight of mix	5.005
Volume of Specimen, cc	1622.85
Weight of Specimen, g	3977.0
Weight of Aggregate, g	3777.951
Weight of Asphalt, g	199.049
% Bitumen Absorption	0.72
Weight of Asphalt Absorbed, g	27.201
Net Weight of Asphalt, g	171.848
Specific Gravity, Aggregate Blend	2.622
Specific Gravity, Asphalt	1.033
Volume of Aggregate (V_m), cc	1440.87
Volume of Asphalt (V_b), cc	166.36
Volume of Aggregate + Asphalt, cc	1607.23
Volume of Air (V_a), cc	15.62
Volume of Voids in Aggregate, cc	181.98
% V	0.96
% V_{ma}	11.21
% V filled	91.42
$e = \frac{V_a + V_b}{V_m}$	0.1263
Length of Specimen, cm	19.90
Cross-Sectional Area, sq. cm	81.55
Aggregate Density, g/cc	2.328
Aggregate Density, pcf	145.267

TABLE 8

Typical Test Data

Type of Test: "Q"											
Specimen: Q-12, 5%, 400 psi											
Loading psi *	Elapsed Time, min.	Vol. Scale cm	δ Vol. Scale, cm	δ Vol. cc	Corr. V, cc	ΔV cc	Dial cm	Δ Dial cm	Piston cc		
$\sigma_3 = 15$	0	37.50		-20.377	-2.465	-2.465	0.043	0.008	-0.023		
	0.25	33.40	-4.10	-1.740	-1.740	-4.205	0.035				
	5.0	33.05	-0.35	-0.248	-0.583	-4.788	0.153	0.118	40.335		
	10.0	33.00	-0.05	1.491	1.272	-3.516	0.236	0.077	-0.219		
$\Delta\sigma = 50$	15.0	33.30	40.30	3.479	3.203	-0.313	0.333	0.097	-0.276		
	20.0	34.00	40.70	4.722	4.469	4.156	0.422	0.089	-0.253		
	25.0	34.95	40.95	6.212	5.945	10.101	0.516	0.094	-0.267		
	30.0	36.20	41.25	10.686	10.302	20.403	0.651	0.135	-0.384		
	35.0	38.35	42.15	17.146	16.580	36.983	0.850	0.199	-0.566		
	40.0	41.80	43.45	25.844	24.991	61.974	1.150	0.300	-0.853		
	43.0	47.00	45.20								
	240										

* Nominal stress on a 4-inch diameter specimen

TABLE 9

Typical Stress and Strain Computations

Type of Test: "Q"
Specimen: Q-12, 5%, 400 psi

Total Load lbs.	Friction lbs.	Net Load lbs.	Δ Vol. cc	$V_1 \neq \Delta V$ cc	Δl cm	$l_1 - \Delta l$ cm	Area sq. cm	Stress psi	Strain %
$\sigma_3 = 15$ psi at 15 sec at 5 min									
			- 2.465	1622.85		19.90	81.55		
			- 4.205	1620.38					
				1618.64					
628	20	608	- 4.788	1618.06	0.110	19.79	81.76	48.0	0.553
1257	30	1227	- 3.516	1619.33	0.077	19.71	82.16	96.3	0.940
1885	46	1839	- 0.313	1622.54	0.097	19.62	82.70	143.4	1.427
2263	59	2204	4.156	1627.01	0.089	19.53	83.31	170.6	1.874
2514	74	2440	10.101	1632.95	0.094	19.43	84.04	187.2	2.347
2765	87	2678	20.403	1643.25	0.135	19.30	85.14	202.9	3.025
2891	95	2796	36.983	1659.83	0.199	19.10	86.90	207.5	4.205
3016	101	2915	61.974	1684.82	0.300	18.80	89.62	209.8	5.533

TABLE 10

Typical Volume Change, Void Ratio and Percent Voids Computations

Type of Test: "Q"

Specimen: Q-12, 5%, 400 psi

Volume of Asphalt 166.36 cc

Volume of Aggregate 1440.87 cc

Stress psi	Δ Vol. cc	Δ Vol. - V_{SV}^* cc	Specimen Vol., cc	Vol. Air cc	% Change in Vol.	Voids %	Void Ratio
15.0		- 2.465	1622.85	15.62		0.96	0.1263
63.0	- 4.205	- 1.740	1620.38	13.15		0.81	0.1246
111.3	- 4.788	- 2.323	1618.64	11.42	-0.107	0.71	0.1234
158.4	- 3.516	- 1.051	1618.06	10.83	-0.143	0.67	0.1230
185.6	- 0.313	- 2.152	1619.33	12.10	-0.065	0.75	0.1238
202.4	- 4.516	- 6.621	1622.54	15.31	0.133	0.94	0.1261
217.9	- 10.101	- 12.566	1627.01	19.78	0.409	1.22	0.1292
222.5	- 20.403	- 22.868	1632.95	25.72	0.775	1.57	0.1333
224.8	- 36.983	- 39.448	1643.25	36.02	1.411	2.19	0.1405
	- 61.974	- 64.439	1659.83	52.60	2.434	3.17	0.1520
			1684.82	77.59	3.977	4.60	0.1693

* Assumes the specimen volume change represented by the 15 sec. reading is due to surface voids only.

friction developed between the piston and the cell. Thus, the total load applied to the specimen was known for each increment of load.

Preliminary tests indicated that volume changes occurred under load to such a magnitude that adjustment of the specimen area was required for the computation of applied stress. This adjustment was made on the assumption that even though the specimen changed volume it would retain a uniform cylindrical shape. This is not strictly the case; however, it is felt that this assumption provided the most reasonable method of computing the average stress acting on the specimen under any given load increment. The average areas were computed in the following fashion:

- (1) The initial volume of the specimen was adjusted in accordance to the volume change that occurred under any load as indicated by the volume measuring device.
- (2) The height of the specimen under the same load was obtained by subtracting the change in height from the initial height of the specimen.
- (3) The average cross-sectional area of the specimen was computed as the quotient of the specimen's volume divided by its height.

Axial strains were determined by dividing the change in height of the specimen by its initial height. In some cases there were slight upward movements of the extensometer during the period of application of the confining pressure only. Strain computations ignored these movements and were based on a zero dial reading obtained at the start of the test.

Typical stress-strain curves are illustrated in Figure 12 for one of the routine test series. All tests resulted in curves of a similar shape. In particular, the curves followed a straight line from zero deviator stress to a point

FIGURE 12. STRESS AND VOLUME CHANGE VS STRAIN

designated as the Proportional Limit (P.L.). Beyond the P.L. the stress-strain curve deviated from the straight line at a rate that increased with axial strain. In approximately one-half of the tests, a peak stress value was obtained which was followed by a decrease in stress with further strain. In tests having this characteristic, pronounced shear planes were observed in the specimen. In the remainder of the tests a bulging of the specimen was noted and a typical plastic failure stress-strain curve resulted. Figure 13 shows a typical specimen before testing and representative failures. The failure planes shown in this figure have been emphasized in order to be visible in the photograph.

All tests were stopped at approximately five percent strain. At this point, the specimens had either developed a shear failure or the slope of the stress-strain curve for bulging failures was so flat as to constitute failure for practical purposes.

Strength and Stability Determinations

The question of how to establish strength and/or stability constitutes a major problem in the testing of asphaltic mixtures. For purposes of this study, deviator stresses at four points were established: (a) at the Proportional Limit, (b) at an axial strain of two percent, (c) at failure or the best estimate of the failure stress where this condition was not clearly established by the stress-strain curve, and (d) at the point where the volume change of the specimen under load was zero.

The results of typical strength and stability determinations are presented in Figures 14, 15, 16, and 17 as plots of vertical stress against lateral stress for conditions of failure and two percent strain for specimens prepared at combinations of 4 and 5 percent asphalt content and 150 and 400 psi compaction pressure. Similar plots were developed for other testing conditions and stability criteria. A typical Mohr's circle plot for one test series is shown in Figure 18 for the failure condition.

FIGURE 13. TYPICAL SPECIMENS BEFORE AND AFTER TESTING

**FIGURE 14. VERTICAL STRESS VS LATERAL STRESS
AT FAILURE - 4% ASPHALT CONTENT**

FIGURE 15. VERTICAL STRESS VS LATERAL STRESS
AT 2% STRAIN - 4% ASPHALT CONTENT

FIGURE 16. VERTICAL STRESS VS LATERAL STRESS
AT FAILURE- 5% ASPHALT CONTENT

FIGURE 17. VERTICAL STRESS VS LATERAL STRESS
AT 2% STRAIN - 5% ASPHALT CONTENT

FIGURE 18. CIRCLES OF STRESS AT FAILURE

Figures 19 and 20 indicate the variation in deviator stress at failure with asphalt content for the various compaction pressures. Also shown in these figures are plots of the variation in bulk density and aggregate density with asphalt content for the different compaction pressures. Data from all routine tests are shown in these figures. For combinations of compaction pressure and asphalt content at which more than one test was performed, the average values of compressive strength, bulk density, and aggregate density are shown in Figures 19 and 20.

Figure 21 presents a graphical summary of the values of the observed shear strength parameters, cohesion "c" and angle of internal friction " ϕ ", at failure for the various asphalt contents and compaction pressures. Average values of the parameters are plotted for conditions at which more than one test series was performed.

Volume Change Characteristics

The computation of the volume of the specimen at any point during the progress of the test was based on observed changes in the volume of the cell liquid as recorded by the volume measuring device and the computation of the initial bulk volume of the specimen. An initial reading of the mercury level in the volume measuring tube provided the reference to which changes in this level could be referred for the computation of volume changes. Readings were obtained at 15 seconds after the application of the confining pressure and at periodic intervals during the testing.

It is assumed that the volume change recorded as the difference in the 15-second and initial readings is due to two separate effects. The first of these is the change in the volume of the apparatus due to the confining pressure. The second is the result of the membrane being forced into the surface irregularities of the specimen.

FIGURE 19. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT - LOW COMPACTION PRESSURES

FIGURE 20. COMPRESSIVE STRENGTH AND DENSITY VS ASPHALT CONTENT - HIGH COMPACTION PRESSURES

NOTE: "C" = cohesion ; "φ" = angle of friction.

FIGURE 21. SHEAR STRENGTH PARAMETERS AT FAILURE VS ASPHALT CONTENT

The volume observed as the difference between the calibration volume value for a given confining pressure and the 15-second reading has been treated as a volume representing surface voids of the specimen. The magnitude of this value is directly related to the amount of the confining pressure and the surface condition of the specimen. In general, the higher the lateral pressure, the greater the measured volume assumed to be surface voids.

The amount of volume change occurring under each successive load increment was computed from the difference in the volume measuring device readings before a load increment application and just prior to the application of the following increment. Therefore, any error induced in the computations due to air in the system, erroneous calibration, personal error in initial reading, etc., is contained within the 15-second reading. The result is, that while the absolute value of the computed volume changes may be slightly in error, the general pattern of these changes and the order of magnitude of the changes are thought to be correct.

The volume of the specimen and the volume of air within the specimen were computed by the algebraic summation of the initial values of these volumes and the incremental changes computed from the differences in the mercury levels between successive readings corrected for the volume displacement due to the movement of the piston into the cell.

Figure 12 shows a typical change in the volume of a specimen under the applied deviator stress as a function of axial strain. For ease in comparing the magnitudes of volume changes between specimens, the changes are presented as a percentage of the initial specimen volume less the volume assumed to represent surface voids.

All specimens showed a change in volume during testing. In all cases there was a decrease in volume followed by an increase in volume. The increase in

volume became linear with axial strain and continued in such a manner to failure. In all cases there was an appreciable increase in volume at the failure condition. The slope of the straight line portion of the volume-change plot varied with confining pressure and with aggregate density. In general, the higher the aggregate density greater the gradient and the higher the asphalt content the lower the volume-change gradient. An increase in confining pressure for tests on specimens having similar initial conditions caused a decrease in the gradient.

The volume changes occurring as a decrease in the volume of the specimen were greatest for the higher lateral pressures. The magnitudes of these changes in any case were small - generally less than one-half of one percent of the reference volume. The greatest decrease in volume occurred for specimens having the lowest aggregate density values.

Void Ratio and Percent Voids

In asphalt mixture terminology, the percent voids (%V) is defined as the amount of air volume expressed as a percentage of the total volume of the specimen. In soil mechanics terms, an expression for void ratio (e) is commonly used to define the ratio of volume not occupied by aggregate solids to the volume of the aggregate solids. Both terms have been used in this study, the former because it provides an easy reference for those familiar with asphalt terminology and the latter because the aggregate solids volume provides an unchanging reference regardless of whether the specimen expands or contracts.

Void ratio and percent voids were calculated utilizing the computed volume of the air and total specimen volume based on observed volume changes and on the known fixed volume of bitumen and aggregate solids in the specimen as computed from the initial measurements of each specimen. Aggregate volumes were computed using the average bulk specific gravity of the aggregate blend and the asphalt volume was

determined from the weight of asphalt in the specimen less the weight absorbed by the aggregate. Sample computations of the void ratio and percent voids are shown in Tables 6 and 10.

Figure 22 shows the void ratio and percent voids plotted against total applied axial stress for a typical test series. The changes in void ratio and in percent voids during a test follow the same general pattern as that of the total volume change. Initially there is a decrease in these values and then an increase to failure. Both terms show a very gradual change in value under the intermediate applications of load. Changes are greatest under the initially applied loads and as the specimen approached failure.

Figures 23 and 24 show the relationships between deviator stress and void ratio and percent voids at the Proportional Limit and at two percent strain for all routine tests performed on specimens prepared at 4 and 5 percent asphalt content at 400 psi compaction pressure. Similar plots were developed for other compaction pressures.

Figures 25 and 26 show similar data to those presented in Figures 23 and 24, but for a wider range in conditions and in a slightly different form. These figures present a plot, for all routine test results, of the deviator stress versus the percent voids at two percent strain. The curves that are approximately vertical represent the deviator stress variation with percent voids at two percent strain for a given confining pressure and compaction pressure. The percent voids data shown in Figure 24 for the two percent strain condition have been plotted also in these figures.

FIGURE 22. PERCENT VOIDS AND VOID RATIO VS STRESS

FIGURE 23. VOID RATIO VS DEVIATOR STRESS - 400 PSI COMPACTION

FIGURE 24. PERCENT VOIDS VS DEVIATOR STRESS -- 400 PSI COMPACTION

FIGURE 25. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN -
LOW COMPACTION PRESSURES

FIGURE 26. PERCENT VOIDS VS DEVIATOR STRESS AT 2% STRAIN - HIGH COMPACTION PRESSURES

DISCUSSION OF RESULTS

Effect of Test Procedure on Observed Strength

The first test series were performed with specimens prepared at 4 and 5 percent asphalt content using compaction pressures of 150 and 400 psi. Tests were performed on similar specimens using three different conditions of drainage. Quick or "Q" tests were performed in which the specimen drainage valve was closed during the application of all load increments. Consolidated -Quick or "Q_c" tests were accomplished in which the drainage valve was open during a one-hour period of confining pressure application and then closed for the application of all axial load increments. The Consolidated-Partially Drained or "CP" tests were completed in a fashion similar to the "Q_c" tests except that the drainage valve was kept open during the application of all loads.

In no case was there a significant difference between the results of "Q", "Q_c", and "CP" tests for specimens prepared at the same asphalt content and compaction pressure. Variation in deviator stress of approximately ten percent were observed but these did not follow a pattern that could be satisfactorily correlated with drainage conditions. It was assumed, therefore, that these differences were random variations due to non-uniformity in the test specimens or to other uncontrolled test errors.

The effect of drainage was found to be equally unnoticeable for tests performed on three specimens using a slow rate of loading. (These data are not included in this paper.) Since the results of the first thirty-nine tests indicated little or no difference in the strength or stability of similar specimens tested under different conditions of drainage, the remainder of the tests were performed using the "Q" test procedure only.

In spite of the fact that drainage conditions did not affect the results, there is considerable evidence to indicate that negative pore pressures did develop within the specimens during testing. An expansion of the specimen under conditions of no drainage implies an increase in the volume of air within the sample and, therefore, a negative pore pressure. It was noted also that if the drainage valve was opened at the completion of a "Q" or "Q_c" test there was an obvious sound of moving air. That this air was moving into the specimen was evidenced by a suction developed on the operator's hand if held over the drainage outlet.

It must be concluded, in light of evidence of negative pore pressure and uniformity of observed strength results regardless of drainage conditions, that, though pore pressures apparently did develop, their effect was so small compared to the strength of the mixture that they did not influence the test results. This conclusion appears to be the only rational one possible for loading times of up to one hour between load increments.

Volume Changes

In every specimen tested, regardless of test procedure or initial conditions, a change in volume of the specimen was noted. All tests followed the same general pattern of volume change. Typical curves of volume change with axial strain are shown in Figure 12. The specimen decreased in volume under the application of the confining pressure with the magnitude of the decrease a function of the magnitude of the confining pressure, all other things being equal. Further, but smaller, decreases in volume occurred under each of the first few axial load increments. Additional axial loading produced a volume increase that continued to failure.

The amount of the reduction in specimen volumes was small. In nearly every case the volume decrease amounted to less than one-half of one percent of the specimen volume less the volume of surface voids. In all cases, the axial strain

at the point of minimum specimen volume was less than that at the Proportional Limit. In nearly all cases, the volume change increased linearly with axial strain over the range of strain values from slightly less than those at the Proportional Limit to values at failure.

The slight reduction in specimen volume noted during the early portion of all tests was thought to be due to several causes. A portion of the reduction was possibly due to the recovery of expansion that may have occurred upon release of compaction pressures applied during specimen fabrication. Another portion of the reduction may have been due to elastic compression of specimens under the confining pressures. In addition, some decrease in volume may have occurred as a result of plastic deformation of the bitumen.

It seems apparent that dilatancy occurs with bituminous mixtures subjected to triaxial shear. This can be explained in a manner similar to that utilized for explaining the same phenomena in dense-granular soils. As shearing begins to occur, grains are forced to move apart in order to ride over one another. This expansion continues until all particles on the shearing plane are free to move. At this point the maximum strength has been reached. The dilatancy or volume increase with axial strain occurs at a uniform rate during shearing and its value is a function of the initial density of the aggregate mass.

A study of the volume change gradients developed between points corresponding to zero percent volume change and two percent axial strain showed the following effects for the bituminous mixtures studied: The magnitude of dilatancy ^{with increasing} increased/compaction pressure regardless of asphalt content. At a constant compaction pressure it was observed that the rate of volume increase with axial strain was greatest for the lower asphalt contents. The former result is as expected based on experience with dense-graded granular soils. The latter observation is not

explained so readily. It appears probable that the magnitude of dilatency is less for the higher asphalt contents because of the presence of thicker films of the viscous binder between particles. This would permit the specimen to accept greater axial strains with less movement of the aggregate particles than if the asphalt film were of less thickness as is the case for the lower asphalt contents. The asphalt would permit plastic deformation of the mixture and would reduce the movement necessary for particles to move over one another.

Relationships Between Void Content and Strength

Computations of the void ratio and percent voids at two percent strain and the corresponding deviator stress show that there is a definite relationship between voids and strength for the mixtures and conditions studied. The rate of movement of the deformation dial and the volume measuring device mercury level were so great at failure that it was felt that void ratio and percent voids computations at this stress would not be reliable. Figures 23 and 24 illustrate this relationship for both void ratio and percent voids for tests performed at all conditions of drainage on specimens prepared at 4 and 5 percent asphalt content and 400 psi compaction pressure.

With limited deviation, the relationship of void ratio and percent voids, both at two percent strain and the Proportional Limit, is linear with the logarithm of the deviator stress. This linear relationship holds for all tests on specimens prepared by the same compaction pressure and at the same asphalt content regardless of confining pressures or drainage conditions during the test. The two percent strain curves indicate that the linear relationship appears to be valid for cases where the values of percent voids and void ratio are both larger and smaller than the initial values of the void ratio and percent voids. Indications are that the slope of the straight line is larger for specimens of the lower asphalt content

at any given compaction pressure than for specimens of the higher asphalt content. Plots of the kind shown in Figures 23 and 24 for other compaction pressures (not included) show that the difference between slopes for the two asphalt contents becomes less at the lower compaction pressures.

In order to verify the uniqueness of the established relationship, two samples were prepared at 4 percent asphalt content and 400 psi compaction pressure and water saturated prior to testing. The strength-voids relationships of specimens fabricated at 4 percent asphalt content and 400 psi compaction pressure are expressed by those of Figures 23 and 24. The water saturation served to reduce the amount of air in the specimens to a quantity essentially equal to that in specimens prepared by the same compaction pressure but at 5 percent asphalt content. Thus the void ratio values of the water-saturated specimens remained nearly equal to those of similar specimens without water saturation, while the percent voids values were reduced as though the asphalt content had been increased. Referring to Figure 24, it is noted that the percent voids - deviator stress relationship for the water-saturated specimens fits the relationships for 5 percent asphalt content very well for both two percent strain and Proportional Limit cases. Figure 23 shows that there is reasonable correlation of void ratio and strength at two percent strain for the water-saturated specimens with that of similar but unsaturated specimens. The relationship of void ratio and strength at the Proportional Limit for the water-saturated specimens shows some deviation from the established curve.

The deviation of the void ratio data for the water-saturated specimens from the curve established in Figure 23 is probably due to the increase in void ratio resulting from the slight increase in specimen volume as a result of water saturation. This increase in volume was considered in the computations of the water-saturated specimens test data, but the effect was to change the initial void ratio conditions slightly from those of specimens not saturated.

The diagonal straight lines of Figures 25 and 26 show the same relationship between percent voids and deviator stress at two percent strain as shown in Figure 24 and also present similar data for routine tests at other combinations of asphalt content and compaction pressure. In addition, Figures 25 and 26 show that apparently there is a unique relationship between percent voids and deviator stress at two percent strain for a given confining pressure and compaction pressure regardless of the asphalt content at which the specimens were prepared.

A study of Figure 26 suggests the possibility of a critical value, with respect to strength, of the percent voids at two percent strain. In particular, the curves for 500 psi compaction are sufficiently curved to indicate the likelihood of a peak value of deviator stress occurring between approximately four and six percent voids. As compaction pressure was decreased the curvature of the plot of percent voids versus deviator stress at a particular confining pressure decreased until a linear relationship is indicated for the 150 psi condition. Unfortunately, there are insufficient data to define clearly the concept of a critical percent voids with respect to strength.

The implication of Figures 25 and 26 is that a family of curves may be developed for a given aggregate, a given asphalt, and an established compaction and triaxial test procedure that will define the deviator stress and percent voids at two percent axial strain. As there is a mathematical relationship between percent voids and void ratio, a similar set of curves could be developed using void ratio in lieu of percent voids.

It should be noted that the above results and conclusions are applicable only to the aggregate, aggregate gradation, and penetration grade asphalt cement used in this study as well as for the particular compaction and testing procedures used. It is felt, however, that similar results for other aggregates, aggregate gradations, and asphalt cements are probable.

Variation of Strength With Asphalt Content

In the interest of expanding the general knowledge of the relationship between asphalt content and compressive strength under conditions of triaxial compression testing, data from all routine tests are presented in a plot between these variables in Figures 19 and 20. Superimposed over the strength curves in these figures are curves expressing the relationship between aggregate and bulk densities and asphalt content.

From Figure 19 it may be noted that the variation in strength with asphalt content at any given confining pressure is quite low for the 150 and 250 psi compaction pressures. In light of the variation in bulk density and aggregate density for the same conditions, this figure provides further verification of the lack of correlation between specimen density and strength.

Figure 20 presents similar data for 400 and 500 psi compaction pressures. The variation in strength with asphalt content at any given confining pressure is noticeable in this figure. Essentially equal strength values were developed under the 400 psi compaction pressure at 3 and 4 percent asphalt contents, but there was a rather sharp decrease in strength at the 5 percent asphalt content condition. This decrease is pronounced even though bulk density and aggregate density show a continuing increase with asphalt content. The 500 psi compaction results again show about the same strength values at 3 and 4 percent asphalt contents for the two lower confining pressures; however, a strength decrease is noticeable from 3 to 4 percent asphalt content for the 60 psi confining pressure. At 5 percent asphalt there is a sharp decrease in strength for 500 psi compaction at all confining pressures. It is apparent also that the sharp decrease in strength at 5 percent asphalt content is reflected by a decrease in aggregate density even though bulk density continued to increase.

On the basis of Figures 19 and 20 it is apparent that compaction pressure greatly affects the observed compressive strength of a specimen formed from a given mixture. It is apparent also that a density criterion for suitability of a mixture from a strength standpoint is not reliable. Furthermore, it appears that the aggregate density of a specimen is a more significant measure of the change of strength than is bulk density.

The apparent shear strength parameters, " c " and " ϕ "; were established by a Mohr's circle of stress analysis for the failure conditions and for the stress at two percent axial strain for all test series. A typical Mohr circle plot for the failure condition is shown in Figure 18. In all cases the envelope of stress was fitted to the circles of stress as a straight line. In some cases the tangency of the circles to the fitted straight line was not perfect. The variation from a straight line envelope, however, was so slight that a curved envelope did not appear to be justified. It is conceivable that further tests at higher confining pressures would emphasize the possibility of a curved envelope of failure.

Figure 21 presents the results of a Mohr's circle analysis of the shear strength parameters at the failure condition for all routine tests. It may be readily observed that there is a decided decrease in the value of the observed angle of internal friction with increased asphalt content at the two higher compaction pressures. This decrease is as expected when it is realized that asphalt film thickness increases with asphalt content and thus tends to reduce the intimate contact between aggregate particles. At the two lower compaction pressures there is little change in the observed angle of internal friction with changes in asphalt content. In fact there appears to be a slight increase in this parameter as asphalt content increases. It is thought that the aggregate density under these compaction pressures is such that an increase in asphalt content serves to

improve compaction efficiency and that a sharp decrease in observed angle of internal friction would occur at still higher asphalt contents where these low compaction pressures are used.

The cohesion parameter relationships shown in Figure 21 follow an interesting pattern. The following comments represent an attempt to explain these relationships utilizing the concepts advanced by other investigators. As an example, see Herrin (11). As compaction pressure increases the variation of cohesion with asphalt content varies gradually from essentially no change to a curve that is markedly convex. For a given asphalt content the value of cohesion increases with compaction pressure for 3 and 4 percent asphalt contents but the cohesion at 5 percent asphalt content decreases upon an increase in compaction pressure from 400 to 500 psi compaction. Apparently the increase in cohesion that occurs with an increase in compaction pressure results from the higher compaction pressure forcing the coated particles into more intimate contact and increasing the contact area between coated particles. If the density of the specimen is increased by increased compaction pressures to such a point that the asphalt tends to develop thicker films between particles, the particles are forced apart and the contact area between the coated particles is reduced. This is apparently the case for the change of compaction pressure from 400 to 500 psi for 5 percent asphalt content. The result is a reduction in cohesion.

For a given compaction pressure an increase in asphalt content acts to increase lubrication and to facilitate compaction. This in turn increases the value of cohesion to a maximum. Beyond the critical value of asphalt content, additional bitumen tends to form thicker films on the particles and to force them apart. The result is a decrease in contact between the coated particles and in the value of cohesion. The foregoing explanation of the effect of asphalt content on cohesion

at a given compaction pressure is clearly illustrated at 400 and 500 psi compaction pressures in Figure 21. The explanation, however, does not appear to apply to the relationship exhibited at the two lower compaction pressures. It is thought that further increases in asphalt content beyond the quantities used in this study at the low compaction pressures would result in a critical value of asphalt content and a decrease in cohesion for asphalt contents beyond the critical value.

It is quite likely that apparent cohesion is not the proper parameter to use for expressing the relationships between cohesion, asphalt content, and compaction pressure. Future studies may permit the establishment of the Hvorslev true cohesion parameter (12) and this concept possibly will provide further understanding of these relationships.

CONCLUSIONS

The results obtained from the analysis of test data from sixty-five triaxial tests performed on specimens prepared at four different compaction pressures and three different asphalt contents appear to justify the following conclusions. It should be realized that these conclusions are applicable only to the particular bituminous mixtures and compaction procedures, and for triaxial compression tests of the types used in this study. Furthermore, it should be noted that all percent voids relationships are based on a consideration of the asphalt available to fill the mineral aggregate voids rather than upon the total asphalt used in mixing.

Differences in the drainage conditions existing during the performance of triaxial compression tests on dense-graded bituminous mixtures do not materially affect strength values. This conclusion appears justified for time intervals between load applications of one hour or less.

Indirect evidence indicates that a negative pore pressure develops in the gaseous phase of a dense-graded bituminous mixture subjected to triaxial compression. The indicated negative pore pressure did not affect the observed strength results, so it must be concluded that the effect of this pressure is so small as to be masked by the high strength values of the mixtures tested.

All test results clearly demonstrate that the bituminous mixture specimens tested changed volume during the progress of a test and that the volume change consists of a slight reduction in volume followed by an expansion to failure.

Within the range of the materials and procedures used in this study, there appears to be a unique relationship between the percent voids and/or the void ratio and the deviator stress at two percent axial strain and at the Proportional Limit. The established relationship apparently depends upon the initial conditions of void ratio and percent voids to which the specimen is compacted. The

results of tests on specimens with the initial conditions varied from the as-compacted values by water saturation appear to justify the established relationship.

Indications of the effect of compaction pressure and asphalt content on the observed apparent shear-strength parameters and on the compressive strength of the test mixtures were established. It was found that variation in measured cohesion values depend upon compaction pressure as well as on asphalt content and that the variation with asphalt content becomes more pronounced at the higher compaction pressures. Similar effects are established for the apparent angle of shearing resistance and for compressive strength at failure. It is clearly shown that variations in bulk density do not necessarily reflect similar variations in strength but there are indications that variations in aggregate density are reflected by similar variations in strength.

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