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STABILITY OF GRANULAR BASE COURSE MATERIALS CONTAINING BITUMINOUS ADMIXTURES

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INTRODUCTION

Soil stabilization for highway base course construction has been defined as "any process aimed at maintaining or improving the performance of a soil as a constructional material, usually by the use of admixtures" (15). The object of soil stabilization is to maintain the soil in a state of high stability, or in some cases, to increase stability. Soil stabilization may be accomplished mechanically by selecting and regulating the gradation of the soil materials or it may be accomplished by adding stabilizing agents, some of which are Portland cement, lime, calcium chloride and sodium chloride.

Bituminous stabilization is but one of the methods currently being utilized to treat soil materials to obtain increased stability and/or to waterproof the soil particles. Rapid depreciation of available gravel and crushed stone suitable for pavement construction without modification, and the rising cost of construction, owing to a need for the use of locally available materials, have led to a significant increase in soil-asphalt stabilization.

Iowa, having over 10,000 miles of primary highways, has encountered problems associated with granular base course materials. The study reported herein was conducted in cooperation with the Iowa State Highway Commission and the U. S. Bureau of Public Roads. Two asphalt constituents, SS-1, a slow setting emulsified asphalt, and a hot mix asphalt cement of penetration grade 120-150, were selected for bituminous stabilization of three major crushed limestone base course aggregates. The asphalt materials were selected in accordance with Iowa State Highway Commission specifications (20). The crushed stone materials were selected in cooperation with the Iowa State Highway Commission's Director of Research, Materials Engineer, and Geologist, as being representative of I.S.H.C. approved crushed stone for rolled stone bases* (18).

Comparisons of the bituminous treatments with the untreated crushed stones are presented in order to show the potential benefits to total stability. The major testing technique used was the consolidated-undrained triaxial shear test with pore water pressure and volume change measurements.

* The terms "aggregate", "stone or stones", and "soil" are used interchangeably throughout this report.

REVIEW OF LITERATURE

The use of asphalts as sealers and as construction materials dates to the primitive era. History books record the first usage of asphalt by the human race during the period from about 3800 to 2500 B.C. The pre-Babylonian inhabitants of the Euphrates Valley in southeastern Mesopotamia, the present Iraq, formerly called Sumer and Accad (Akkad), and later Babylonia, allegedly utilized asphalt as a waterproofer, or sealer, and as a cementing agent. It also was utilized in forming ornaments such as rings and other types of jewelry. The ancient Sumerian, Sanskrit, Assyrian, Accadian, Hebrew, Arabic, Turkish, Greek, and Latin languages have vocabulary words which mean asphalt or bitumen.

The Bible records in the Book of Genesis (Genesis VI, 4) that the Ark, constructed by Noah, was treated with "pitch" within and without: "Bituminabis eam bituminae". Many people contend that this passage indicates Noah used asphalt in the construction of the Ark, approximately 2500 B.C. History books record the use of asphalt as a waterproofer and sealer for the canoes and dugouts of primitive tribesmen, and even today, asphalt is utilized in a similar manner.

During primitive times, asphalt was primarily used as a waterproofer and cementing agent. Nebuchadnezzar gave his father, King Nabopolassar, the credit for constructing the first asphalt block pavement in Babylon during the period 625 to 604 B.C. During the era of King Nebuchadnezzar (604 to 561 B.C.) a street was constructed of stone slabs set in a bituminous mortar, the interstices being narrow at the surface and widening towards the base of the stones. Many modern pavements, composed of stone blocks set in asphalt, are similar to the street built by Nebuchadnezzar, though the art of this construction was lost from approximately 600 B.C. until its rediscovery in the nineteenth century A.D.

In 1835 A.D., it is recorded that on June 15 the first asphalt mastic foot pavement was laid at Pont Royal, Paris, France. The use of Seyssel asphalt for pavements was introduced in England by the French in the year 1836, and in 1838, Seyssel asphalt was introduced into the United States for sidewalk construction in Philadelphia, Pennsylvania.

The first modern asphaltic roadway was constructed in France in 1852. Many people give a Belgian chemist, E. J. De Smedt credit for constructing the first rock asphalt roadway in the United States which consisted of a short experimental highway in Newark, New Jersey, in 1870. In 1876, Pennsylvania Avenue in Washington, D. C., was constructed of Trinidad asphalt (1).

Since its introduction into the United States as a road material, asphalt has been used for city streets, secondary and primary highways, and recently has been utilized for the construction of turnpikes and major interstate highways. To date, few articles have been published which relate data pertaining to laboratory and in-place testing of bituminous base course stabilization. Foster (11) contends that asphalt materials vary so much in stress distribution ability due to rate of loading and temperature, that the effectiveness of stability can be shown only through roadway condition studies during a period of one year. Thus, much of the information available is based upon actual experience with asphalt stabilized soils.

A further illustration of the lack of scientifically controlled experimentation with soil-asphalt base mixtures follows. About 1932, American interests in the feasibility of soil-asphalt stabilization arose in a number of states. Since bitumen stabilization is affected by numerous factors, each state which used bituminous bases developed its own tests and specifications. Though the various states have introduced their numerous tests and specifications for consideration, and while the American Society for Testing and Materials is constantly formulating and standardizing test procedures, only one method of test for soil-bituminous mixtures is standardized by ASTM, i.e., Designation D915-61, titled "Soil-Bituminous Mixtures" (5).

Much confusion also arises concerning the purpose of asphalt as a stabilizing agent. Bituminous material may be added for one or two primary purposes. First, the asphalt will act as a waterproofing agent for the soil particles, and second, the asphalt will act as a binding agent. Most published literature is concerned with the latter concept, that of the cementing properties. Baskin and McLeod (7), presented a concept based solely on the waterproofing characteristic of bituminous material. Their design incorporates the aggregate used for base course construction, waterproofed by asphalt cement in the proportion of one to two percent by weight. They have termed this type of construction "Waterproofed Mechanical Stabilization". The asphalt cement acts solely as a waterproofing agent and the strength characteristic is derived from the mechanical stabilization of the aggregate mixture. The "Waterproofed Mechanical Stabilization" construction for base course material is comparable with the bituminous "soil stabilization" procedures of the Iowa State Highway Commission (20) in which asphalt cement is utilized for waterproofing and binding in the proportion of four percent by weight of aggregate materials.

Asphalt is a complex organic compound which occurs as a product of nature or is derived from the fractional distillation of petroleum crude. The initial distillation provides products, which include (1) straightrun gasoline, (2) kerosene distillate, (3) diesel fuel, (4) lubricating oil, and (5) heavy residual material. The heavy residue, which is of the SC-0, or slow curing road oil consistency, is further treated to provide asphalt cements, road oils, kerosene cutback asphalts, gasoline (naphtha) cutback asphalts, and emulsified asphalts. Each of these is refined to provide bituminous products of various rates of cure (21).

Wilkinson and Forty (33) describe emulsified asphalt as a liquid product in which a substantial amount of asphaltic bitumen, or other bituminous road binder, is suspended in finely - divided condition in an aqueous medium by means of one or more suitable emulsifying agents. When emulsified asphalt is utilized for the stabilization of the aggregate mixture, the water or aqueous media will evaporate, and the remaining asphalt adheres to the aggregate particles acting as a binder and waterproofing agent. Advantages of using emulsions for stabilization, include the ease with which the liquid penetrates into small cavities, the ease of surface coating and the ease of application. A major deterrent to the use of emulsions is the difficulty of storage. In cold weather, the water may freeze and the emulsion will no longer be of use. If the emulsion is allowed to stand in its container without mixing for a prolonged period of time, there is a possibility that the asphalt will coagulate and the mix is rendered useless. Emulsions which obtain their maximum cure after varying time periods may be obtained depending upon the construction requirement.

The first asphalt emulsion patent was obtained in 1903 for the successful use of emulsified dust-laying oils. The credit for pioneer work on emulsions is given to British invention, although America did much to further the investigations. The original emulsion patents were comprehensive in their claims, however most literature indicates that emulsions have been utilized primarily for construction of surface courses.

There have been very few articles published which are concerned solely with emulsified asphalt stabilization of base course aggregates. Since 1924, however, emulsions have been obtainable which suit practically every climate and which are adequate for almost every phase of road-making (12).

A recent presentation by Dunning and Turner (9) provides a collection of laboratory procedures of tests related to the "Evaluation System" for soils stabilized with asphalt emulsion. Data obtained during the development of the evaluation system show that the best R values (an R value procedure was developed by the state of California which indicates effectiveness of the stabilization, and was altered somewhat by Dunning and Turner) are obtained when a stabilized soil is compacted at a total liquid content of 1-3% less than optimum moisture content of the untreated soil. It was also noted that the maximum density of the stabilized soil is often greater than the maximum density of the untreated soil compacted under similar conditions. Dunning and Turner believe that the water in the emulsion is "wetter" than the water normally used for compaction due to the presence of surface active emulsifying agents.

The foregoing investigation introduces one of the primary benefits for emulsion use. With heated asphalt and aggregate mixtures, the stone must be dry or the mixture will foam. The controlled process of foamed asphalt soil stabilization apparently is of beneficial use (8). However, if uncontrolled, foaming occurs due to moisture present in the soil, and the quality of asphalt stabilization is reduced. Unlike the heated product, emulsion stabilization depends upon the presence of moisture in the aggregate. For each soil, or aggregate material, there is an optimum moisture content for maximum dry density. With regard to field compaction of base course materials stabilized with emulsions, Martin and Wallace (21) state that compaction of soil-asphalt mixes retaining relatively high percentages of moisture and volatiles results in base courses of low stabilities. Therefore, the mixtures must be aerated to reduce the amount of both moisture and volatiles. Aeration is accomplished by manipulating the material to encourage evaporation. The moisture content should be reduced to approximately three-fourths of optimum moisture for the mix prior to applying the compactive effort.

The American Society for Testing and Materials (5) Designation D8-63, describes asphalt as a dark brown to black cementitious material, solid or semisolid in consistency, in which the predominating constituents are bitumens which occur in nature or are obtained as residuals in refining petroleum. Asphalt cement is termed as a fluxed or unfluxed asphalt specially prepared as to quality and consistency for direct use in the manufacture of bituminous pavements, and having a penetration at 25°C (77°F) of between 5 and 300, under a load of 100 grams applied for 5 seconds. Asphalt cement may be used to waterproof base course aggregates and to act as a cementitious binder. A major disadvantage associated with the use of this material for base course stabilization is that the asphalt and aggregate must be heated before they are mixed. ASTM Designation D1663-64 (5) specifies temperatures for mixing plant operations as no greater than 300°F for asphalt cement and between 250° and 325°F for the dried aggregate at time of blending. A major advantage associated with this stabilization method, however, is that the only curing period required, unlike the cutbacks and emulsions, is the time for cooling. Thus asphalt cement stabilization is beneficial for use on jobs requiring speed of construction. The Iowa State Highway Commission (20) specifies a 120-150 penetration hot asphalt for stabilization of base course materials.

Harvey (16) lists ten advantages for stabilizing base course materials with asphaltic products:

1. Compared with granular bases of the same thickness, asphaltic compounds reduce the traffic stresses imposed on the subgrade.
2. Asphalt stabilized bases need not normally be as thick as granular bases, thus reducing the total thickness design for the pavement structure.
3. Local materials of a quality not satisfactory for standard granular bases may be employed.

4. Construction delays due to bad weather are held to a minimum since asphalt bases may be laid rapidly by machine and consolidated promptly, making them at once watertight and usable.

5. They protect the subbase from rain and permit haul traffic to use the roadway without damage.

6. Asphalt bases may be opened to traffic for a year or more before any surfacing is laid, allowing full time for possible settlement.

7. They require no protection from frost.

8. They prevent capillary moisture and water vapor from accumulating in the pavement courses where high strength is required.

9. Asphalt bases have uniformity which varies little from place to place.

10. Machine-laid asphalt bases appreciably improve the riding qualities of the final surfacing.

Asphalt cement has been utilized as a base course stabilizing agent for the last half century. Recent tests conducted by Warden and Hudson (32) present the following conclusions pertaining to hot-mixed black base construction with natural aggregates:

1. A wide range of gradations of sand-gravel aggregate may be used. Practical limits for percent passing and Job-Mix Formula tolerances are:

Sieve Size	Percent Passing	Job-Mix Formula Tolerance
No. 4	45-75	6%
No. 20	20-50	4%
No. 200	2-8	1%

2. As the lower courses of the pavement do not reach temperatures as high as the surface, Marshall stability at 140°F is not critical. However, stability of 500 pounds appears to be a practical minimum value for this type of construction. Flow should be less than 0.14 inches.

3. There has been some indication of a plastic condition developing in the lower course of the asphalt bound base, both during construction and under traffic, when high asphalt contents are used. To provide adequate protection against surface rutting it is advisable to maintain total voids at 5 to 7 percent for both sand and sand-gravel mixtures. The acceptable range of voids filled with asphalt appears to be 60-70 percent for sand-gravel and 65-75 percent for sand mixtures.

4. The natural fillers occurring as minus No. 200 material in aggregate deposits should be tested in advance. Natural fillers which have a pronounced effect on penetration and ductility of the filler-bitumen mortar should be avoided.

5. Field experience indicates that due to the softening effect of solvents and solvent vapors on asphalt bound bases, emulsions rather than cut-backs should be used for tack coats.

6. Economical and satisfactory black base mixtures can be produced

using a wide range of local materials. Further economics may result if it can be demonstrated that under actual highway conditions black base can be substituted for thick courses of other types of base construction.

The triaxial test procedure for investigating the strength parameters of bituminous stabilized soils and base course mixtures was introduced by an organization known as the "Triaxial Institute" which later became a committee of the ASTM. Hveem and Davis (19) have quoted from a 1947 letter by C. V. Kiefer of the Shell Oil Company, San Francisco, California, regarding the formation of the "Triaxial Institute":

> "Presently, several separate organizations utilize triaxial testing in one form or another but no two are exactly parallel in all respects. If standardized procedure, units of measure, and nomenclature can be secured by cooperative test, discussions and evaluation and comparison of data, a great service will have been rendered flexible pavement design. Such, in brief, will be the purpose of the organization."

The initial meeting of the "Triaxial Institute" was held in May, 1948. In October, 1949, the group met as an ASTM project committee.

Endersby (10) presented a comparison of the Mohr-Coulomb theory of triaxial testing of dry aggregates and bituminized aggregate. He explains that the triaxial test develops the fact that there is a certain lateral pressure at which every aggregate will carry the same vertical load whether dry or bituminized. This follows from the fact that friction goes down and cohesion goes up as bitumen is added so that the two envelopes cross. His illustration of the Mohr analysis indicates a curved, rather than a straight, envelope of failure for the test of both dry and bituminized aggregate.

In a 1948 presentation, McLeod (24) based on studies by Holtz, Nijboer, and Rutledge (17, 25, 27) assumed a straight line envelope of failure as provided by the Mohr analysis to develop equations of stability and stability diagrams, for purely cohesive and purely granular materials, and for materials having both cohesive and granular properties. He stated that the Mohr diagram provides a fundamental basis for defining the term "stability" as applied to granular and cohesive materials in general, and to flexible base course and surfacing materials in particular. In the presentation of a later study he stated that "It is accepted as experimental fact that most bituminous paving mixtures have Mohr envelopes that appear to be essentially straight lines, but that for some the Mohr envelope is curved" (23).

In their investigations of the triaxial compression method of test for soils stabilized with emulsions, and other asphaltic materials, Oppenlander and Goetz (26) indicate a bituminous-aggregate mixture is a three-phase system with properties not unlike those of a granular soil mass. Under the action of a loading system, it was observed that the behavior of a bituminous-aggregate mixture is more nearly in accordance with Mohr's theory of strength than with any other strength theory. It was stated that Mohr's analysis provides a basic and logical approach to the evaluation of the strength of bituminous-aggregate mixtures.

Smith (28) explains the choice of a specimen of 4 inches diameter and 8 inches height as being suitable for bituminous testing due to the fact that all aggregates normally encountered in bituminous paving can be handled. The specimen height of 8 inches is employed in order to eliminate the effects of interference of shear cones and friction against the testing head.

Smith further presented a discussion of the triaxial testing procedure in comparison with other stability tests. He explained that unconfined compression, Hubbard - Field, Marshall, and numerous extant punching shear tests usually indicate that maximum stability of a mixture is obtained with that combination of asphalt and aggregate providing a compacted mix of maximum density. Triaxial test results indicated achievement of maximum cohesion at the same asphalt content yielding maximum density. However, the friction angle was seriously reduced at the points of combined maximum density - maximum cohesion. He concluded that the maximum compressive resistance of the mix occurs at an asphalt content less than that required for maximum density, and thus, for maximum criteria set forth in the other stability test procedures indicated above.

Goetz and Chen (14) arrived at similar conclusions through triaxial testing. Maximum stability for the asphalt-aggregate mixtures investigated occurred at an asphalt content less than that required for maximum density. Contrary, however, to Smith's investigation, Goetz and Chen conclude that maximum cohesion is produced in a mixture at an asphalt content less than that at which maximum density occurs, and that maximum stability depending on aggregate gradation, occurs at or near maximum cohesion. They further conclude that the angle of internal friction decreases as asphalt content increases, but is not significantly influenced by the penetration grade of asphalt cement. Cohesion, however, decreases as the penetration increases.

A further comparison between asphalt-aggregate mixtures produced from crushed limestone and a gravel of the same gradation was made by Goetz and Chen. When compared to the gravel mixture, the crushed limestone mixture provided increased values for the internal friction angles and cohesion.

Aldons, Herner and Price (2) reported on triaxial tests of non-bituminous stabilized base course aggregate. With regard to compaction processes, their data indicates that vibratory compaction of granular base materials provides the most adequate test specimens. Furthermore, if the physical characteristics of the specimens are kept within reasonable limits, the mean deviation in strength from the average of a large group is less than ten percent. Their data also indicates that the strength and deformation characteristics of a given material depend primarily upon density, with moisture and gradation exerting secondary influences.

Goetz (13) compared triaxial and Marshall test results by first forming and testing Marshall samples. For triaxial testing, he attempted to achieve equivalent Marshall densities in three by seven and one-half inch high specimens which were formed by rodding and statically compacting the materials in a double-plunger floating cylinder using a load of about 5000 psi and maintained for one minute. It was apparent, though the temperatures were held constant during molding by the two methods, that inherent differences in aggregate arrangement, etc., occurred. There was no direct comparison of physical properties obtained in each test as results were expressed in different units. Thus, comparisons were made by evaluating the variables incorporated into the study as affected by each test method and by comparing asphalt contents selected for design of mixtures through each method. He concluded that within the range of materials, mixtures, and specific testing methods used in his study, it appeared the Marshall test would provide the same general qualitative evaluation of an asphalt - aggregate mixture as the triaxial test. Quantitatively, the results differed with regard to mix variables. He noted that the triaxial test is an excellent research tool but the Marshall test appears adequate for the design and control of bituminous paving or aggregate mixtures.

In concluding this literature search, the words of McLeod (22) appear most appropriate:

> "At the present time, the art of designing bituminous mixtures is far ahead of the science. Consequently, the tests in most common use, Hubbard-Field, Marshall, and Hveem Stabilometer, that have been developed in an attempt to indicate the stability of bituminous paving mixtures in service, are of a strictly empirical nature."

MATERIALS

Three crushed stones and two bituminous additives were utilized in this study. Each crushed stone was selected in cooperation with the Iowa State Highway Commission's Director of Research, Materials Engineer, and Geologist, as being representative of the Commission's approved crushed stone for rolled stone bases. The three materials are described in a report by Hoover (18):

1. A weathered, moderately hard limestone of the Pennsylvania system, which outcrops about half of the state of Iowa. Obtained from near Bedford, Taylor County, Iowa. Hereafter referred to as the Bedford sample.

2. A hard limestone of the Mississippian system, obtained from near Gilmore City, Humboldt County, Iowa. Hereafter referred to as the Gilmore sample.

3. A hard dolomite of the Devonian system, from near Garner, Hancock County, Iowa. Hereafter referred to as the Garner sample.

Representative samples of each of these stones were ground to pass the No. 100 U. S. standard sieve. Part of each sample was used for X-ray mineralogical identification and the remaining portion for quantitative measurement of pH, cation exchange capacity, and hydrochloric acid soluble and non-soluble minerals. The chemical and mineralogical test results are presented in Tables 1, 2 and 3.

Physical and engineering property tests were run on whole samples of each stone in accordance with standard ASTM test procedures. Table 4 presents the results.

Table 1. Mineral constituents of the whole material by X-ray diffraction

Stone Des.	Calcite	Dolomite	Quartz	Feldspars	Calcite/Dolomite Ratio*
Bedford	Pred.	Small Amount	Trace	Not Ident.	25
Garner	Pred.	Second Pred.	Trace	Not Ident.	1.16
Gilmore	Pred.	None	Trace	Not Ident.	

* Obtained from X-ray peak intensity

Table 2. Non-HCl acid soluble clay mineral constituents of the whole material by X-ray diffraction

Stone Des.	Mont.	Vermiculite Chlorite	Micaceous Material	Kaolinite	Quartz
Bedford	None	Not Ident.	Pred.	Poorly Crystalline	Large Amount
Garner	None	Small Amount	Pred.	Second Pred.	Large Amount
Gilmore	None	None	None	Pred.	Small Amount

Table 3. Quantitative chemical analysis of whole material

Stone Des.	pH	CEC, (me/100.0g)	Non-HCl Soluble Clay Minerals, %	Non-clay Mineral, Non-HCl Soluble Mat'l., %	HCl Soluble Calcareous Material %
Bedford	9.40	10.88	10.92	Trace	89.08
Garner	9.25	10.60	5.70	1.03	93.27
Gilmore	8.99	5.86	<1.66	Trace	>98.34

Table 4. Representative engineering properties of crushed stone materials

	Bedford	Garner	Gilmore
Textural Composition, %			
Gravel (2.00 mm)	73.2	61.6	66.8
Sand (2.00 - 0.074 mm)	12.9	26.0	23.3
Silt (0.074 - 0.005 mm)	8.4	10.2	5.9
Clay (0.005 mm)	5.5	2.2	4.0
Colloids (0.0001 mm)	1.7	1.4	0.9
Atterberg Limits, %			
Liquid Limit	20.0	Non-	Non-
Plastic limit	18.0	Plastic	Plastic
Plasticity index	2.0		
Standard AASHO - ASTM Density:			
Optimum moisture content, % dry soil weight	10.9	7.6	9.4
Dry density, pcf.	127.4	140.5	130.8
Specific Gravity of Minus No. 10 sieve fraction	2.73	2.83	2.76
Textural Classification		Gravelly Sandy Loam	
AASHO Classification	A-1-b	A-1-a	A-1-a

The stabilizing agents utilized for this study were a slow setting emulsion, SS-1, and an asphalt cement, 120-150 penetration. Both were selected in accordance with the specifications of the Iowa State Highway Commission (20) and were certified by the manufacturers, as meeting the requirements of the Commission.

The Commission specifies that the SS-1 emulsion will meet the requirements of AASHO specification M-140-64-I (4). Typical properties of this material are listed below:

TYPE	SLOW SETTING	
GRADE	SS-1	
	Min	Max
Viscosity, Saybolt Furol at 77F (25C) sec.	20	100
Settlement (*) 5 days		5
Cement mixing test, percent		2.0
Sieve test, percent		0.10
Residue by distillation, percent	57	
TESTS ON RESIDUE FROM DISTILLATION TEST		
Penetration, (77F) 25C, 100 g, 5 sec.	100	200
Ductility, (77F) 25C, cm.	40	
Soluble in Carbon Disulfide:		
Petroleum asphalt, percent	97.5	
Native asphalt, percent	95.0	
Ash, percent		2.0
SUGGESTED USES	Plant or road mixture with graded and fine aggregate with a substantial quantity passing a 1/8-inch sieve and a portion may pass a no. 200 (74 micron) sieve. Slurry seal treatments.	

(*) The test requirement for settlement may be waived when the emulsified asphalt is used in less than five (5) days time; or the engineer may require that the settlement test be run from the time the sample is received until it is used, if the elapsed time is less than 5 days.

The Iowa Highway Commission specifies that the asphalt cement, 120-150 penetration, will meet AASHO specification M-20-63-I (3) with the exception that the loss in weight on heating in the thin-film oven test shall not exceed 0.75 percent for the 120-150 penetration grade. Typical properties of the asphalt cement are listed below:

Penetration Grade	120-150	
	Min	Max
Penetration at 77F, 100g, 5 sec.	120	150
Flash Point, Cleveland open cup, F.	425	---
Ductility at 77F, 5cm. per min., cm.	100	---
Solubility in carbon tetrachloride, percent	99	---
Thin-film oven test, 1/8 in., 325 F, 5 hours:		
Loss on heating, percent	---	1.3
Penetration of residue, percent of original	46	---
Ductility of residue at 77F, 5 cm. per min., cm.	100	---
Spot test (when and as specified, see Note 1) with:		
Standard naphtha solvent	Negative	
Naphtha-xylene solvent, percent xylene	Negative	
Heptane-xylene solvent, percent xylene	Negative	

Note 1 - The use of the spot test is optional. When it is specified, the engineer shall indicate whether the standard naphtha solvent, the naphtha-xylene solvent, or the heptane-xylene solvent will be used in determining compliance with the requirement, and also, in the case of the xylene solvents, the percentage of xylene to be used.

METHODS OF TESTING

Three methods of specimen compaction and two methods of specimen tests were used in this study. The major testing procedure was triaxial shear, which provided data on the angle of internal friction, cohesion, volume change, pore water pressure and strain characteristics of the bituminous stabilized crushed stone mixtures.

Compaction

To determine the optimum moisture content and maximum dry density of the stones treated with SS-1 emulsion, the standard Proctor compaction test was used. For determining the average dry density of the specimens treated with hot asphalt cement, the Marshall compaction method was utilized. And for all of the specimens compacted for the triaxial shear test, the vibratory compaction method was determined to be the most satisfactory.

Standard Proctor compaction

The standard Proctor compaction method (5) was used to provide data on the optimum moisture content and maximum dry density of the three aggregates stabilized with asphalt emulsion. Each aggregate was air dried prior to preparing the treated mixtures. Two samples of the air dried stone were used to obtain the hygroscopic moisture of the aggregate. This procedure was repeated for each air dried sample.

Sufficient aggregate to fill a Proctor mold was weighed to the nearest 0.1 gram and placed in a large mixing bowl. Distilled water was added in proportionate amounts, which, in conjunction with the water of the emulsion, was sufficient to provide data for plotting the moisture density curve. The aggregate and water were mixed by hand to insure maximum coverage of aggregate particles.

An SS-1 emulsion contains 60 percent residual asphalt. The remaining 40 percent of the emulsion is an aqueous solution of water and emulsifier which contributes a portion of the water required for optimum moisture content of the sample. According to Iowa State Highway Commission specifications, the emulsion is to be added to the aggregate mixture in quantities sufficient to provide residual asphalt in the amount of 3 percent, by weight of the dry aggregate. For example, if a 500 gram sample of aggregate was weighed for testing, 25 grams of the liquid emulsion was added to the mix. Asphalt emulsion has a specific gravity of approximately one, thus the 25 gram liquid sample contains 10 grams of water and 15 grams of residual (3% by weight of the dry aggregate).

After sufficient time had elapsed to allow the distilled water to penetrate into the aggregate voids, the measured sample of asphalt emulsion was added to the mix. Again the mix was stirred by hand to insure maximum coverage of the aggregate particles. The mixture was then placed in the mold and compacted according to ASTM Designation D698-64T, "Test for Moisture-Density Relations of Soils, Using 5.5-lb. Rammer and 12-in. Drop (Tentative)" (5).

After compaction, each specimen was weighed in the mold, extruded, broken, placed in the drying oven, and allowed to dry to constant weight. Total moisture content of each specimen was then determined.

The procedure outlined was repeated, varying only the amount of distilled water added to the mix, until sufficient data was provided to determine the moisture-density curve indicating optimum moisture for maximum dry density of the mix.

Marshall compaction

To obtain the average dry density of the samples treated with hot asphalt cement, the Marshall compaction procedure was utilized (5). Specimens were prepared in the following manner:

1. Large pans of aggregate were placed in an oven and allowed to come to an equilibrium temperature of $310^{\circ} \pm 10^{\circ}F$. This temperature was selected to correspond to the requirement established for aggregate temperatures at a mixing plant (5).

2. The asphalt cement was maintained at a temperature of $270^{\circ} \pm 10^{\circ}F$ in a 2 quart pouring can placed on a temperature regulated hot plate.

3. The heated aggregate was removed from the oven, and a portion was weighed and placed into a large mixing bowl.

4. The heated asphalt was then weighed to provide a treatment of 4%, by weight of the dry aggregate.

5. The measured sample of asphalt cement was poured over the hot aggregate, and the mix was stirred by hand. Hand mixing was utilized for all specimen preparation to insure maximum coverage of all aggregate particles. Mix temperature was maintained in excess of $225^{\circ}F$ prior to compaction.

6. The heated asphalt-aggregate mixture was placed into a heated Marshall compaction mold and compacted in accordance with ASTM Designation D1559-62T (5). Six, or more, specimens of each aggregate

treated with asphalt cement were molded in order to obtain an average dry density.

7. Dry densities were determined by weighing and measuring the height of each specimen. An alternate method of weighing the sample in air and then in water was not used, because a more precise correlation of densities of Marshall and triaxial test specimens could be determined by the dry weight method.

Vibratory compaction

Selection of the vibratory compaction method for preparing triaxial test specimens was based on a study conducted in the Iowa State University Soil Research Laboratory (18).

The compaction apparatus consists of a cylindrical mold and a Syntron Electric Vibrator table (Figure 1), which was operated at a 3600 cycle per minute frequency, an amplitude of 0.368 mm., for a 2 minute period. A surcharge weight of 35 pounds was used for compaction of the emulsion treated specimens to obtain standard Proctor densities. The surcharge weight was increased to 125 pounds to obtain densities of the asphalt cement treated triaxial specimens which would parallel the densities obtained by the Marshall compaction method. The vibratory compaction method produced uniform densities while minimizing degradation and segregation of the three stones tested.

Aggregate-emulsion and aggregate-asphalt cement mixtures were prepared for vibratory compaction in the same manner as they were prepared for the Proctor and Marshall compaction molds, with one exception; i.e., 500 grams excess of the aggregate-emulsion mixture, beyond that required for the vibratory mold, was prepared for moisture

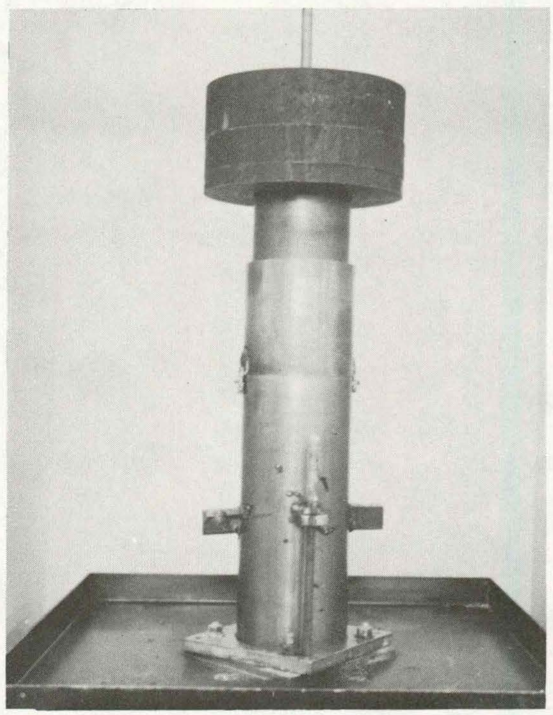

Figure 1. Vibratory compaction apparatus

content determinations.

The aggregate-emulsion mixture was placed in the vibratory compaction mold in three equal layers, each layer being rodded 25 times. The weight was set in place on top of the specimen and compaction was completed as indicated above. The 4 inch diameter by 8 inch high specimen was then removed from the mold, weighed, measured, and placed in the drying room to cure to a constant weight. The excess 500 grams of mix was used to determine the sample moisture content at the time of compaction.

The aggregate-asphalt cement mixture, heated to a temperature in excess of 225°F, was placed in a heated vibratory mold, and the mixture was compacted in the same manner as that of the aggregate-emulsion mixture. Dry density was determined by weight and height measurements of the specimen following cooling to room temperature.

Extreme care was taken to insure that the densities of the vibratory compacted specimens treated with emulsion were within 2.0% of standard Proctor maximum dry density and the densities of the specimens treated with asphalt cement were within 1.5% of the predetermined Marshall dry density, for each of the stones tested.

Triaxial Shear

Two basic tests were used to analyze the bituminous stabilized crushed stone materials. All specimens treated with emulsion were tested by triaxial shear. All asphalt cement treated specimens compacted by the vibratory method were tested by triaxial shear, while specimens molded by the Marshall procedure were tested in the standard Marshall test apparatus.

The aggregate gradation of each of the three stone samples, as noted in the materials portion of this report, was selected for treatment and test with emulsion. The same aggregate gradations were selected for test with the asphalt cement, however, an additional set of tests was conducted on a Bedford sample which had been dry sieved to remove the material passing the No. 200 U. S. Standard sieve.

The triaxial shear test machine used in this study was constructed at the Iowa State University Engineering Shop according to specifications of the I. S. U. Soil Research Lab. The unit consists of two bays which can be used to test two specimens simultaneously under different lateral pressures (Figure 2). The apparatus is designed for 2.8 inch by 5.6 inch and 4.0 inch by 8.0 inch cylindrical specimens under all normal triaxial test conditions. The base pore water pressure of the specimen was recorded with a Karol-Warner pore pressure device, designed to measure positive and negative gage pressures. Volume change of the specimen during testing was measured with a water column in a graduated tube connected to the base of the cell and raised or lowered manually to counteract volume changes within the test cell. Precision of the volume measurement device is $\pm$ 0.01 cu. in.

The lateral pressure for specimen testing ranged from 5 to 80 psi, though the majority of the specimens were tested at 10, 20, 30, 40, 60, and 80 psi with additional tests at interim pressures to check any discrepancies in data. Lateral pressures were applied by an air over deaired distilled water system.

Prior to applying the axial load, specimens were allowed to consolidate, with drainage, under the applied lateral pressure until a

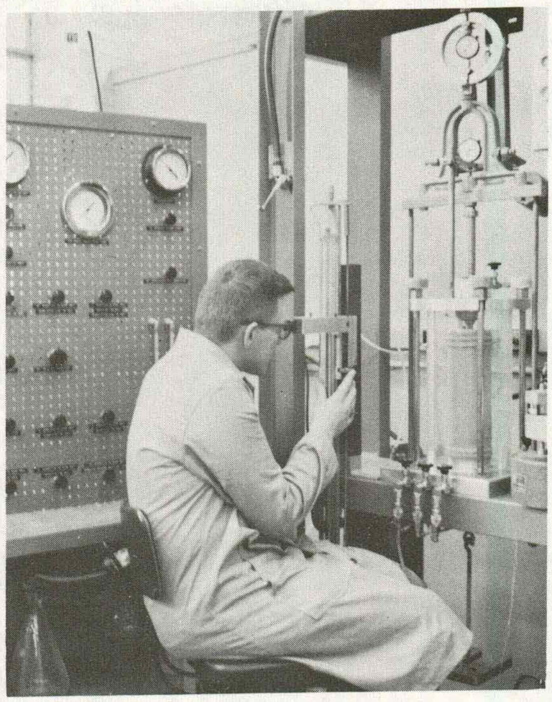

Figure 2. Triaxial shear test apparatus and control panel

constant volume was obtained. Specimens were then sheared under axial load at a deformation rate of 0.01 inch per minute to well beyond actual failure of the specimen. Proving ring load, volume change and pore water pressure readings were recorded at vertical deformation intervals of every 0.010 and/or 0.025 inch.

The asphalt emulsion treated specimens were tested in the cell containing distilled water at room temperature. A special test procedure, however, was used for the asphalt cement treated specimens at a temperature of 100°F. The latter specimens were first stored in an oven maintained at 100°F, for a minimum of 12 hours, to insure thorough heating of the material. Following placement of the specimen and set-up of the triaxial device, the plexiglass cell was wrapped with heating tapes connected to a Powerstat voltage control (Figure 3). The cell and tapes were then surrounded with a fiberglass insulation cover to insure maintenance of the cell water at 100°F (Figure 4). The cell cap was specially fitted with a sealed connection allowing thermocouple wires to be submerged into the water. The other ends of the thermocouple were connected to a potentiometer allowing temperature to be read directly in degrees centigrade. Temperature was manually maintained at 100°F by adjusting the Powerstat. Additional heat tapes were placed on the water storage reservoir. By maintaining the water in the reservoir at 100°F, the need for a waiting period to obtain equilibrium temperature in the test cell was eliminated.

Data obtained during the triaxial shear test were written in computer form for a specially prepared IBM 7074 program. The computer program was then used in a 1627 plotter which graphed effective

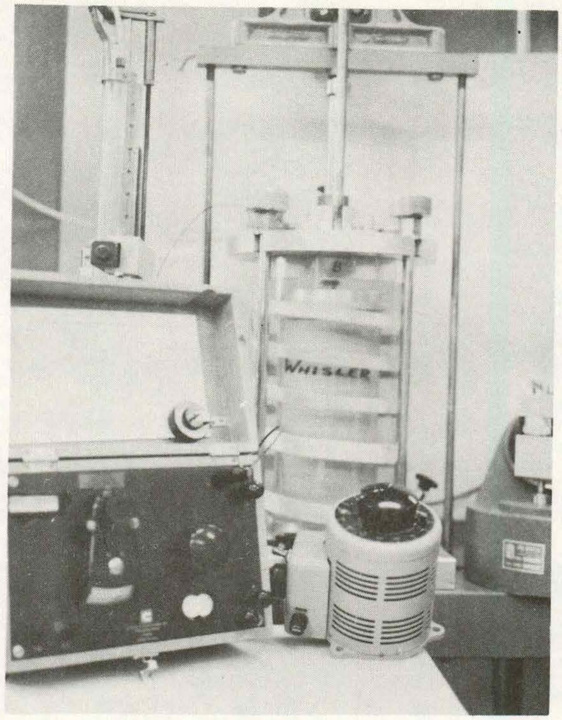

Figure 3. Triaxial test cell with heating tapes, potentiometer, and Powerstat voltage control

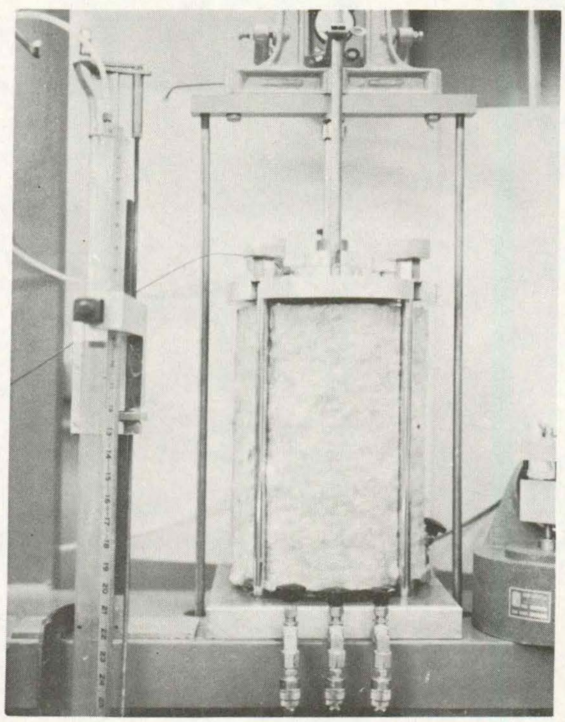

Figure 4. Heated triaxial test cell with fiberglass insulation

stress ratio, % volume change, and pore pressure versus percent strain for each specimen tested. Both readout and graphical data were then used to analyze the test results.

Marshall Stability

The Marshall Stability Test was performed on the asphalt cement treated specimens prepared as previously noted. Each specimen was first placed in an oven maintained at $100^{o}F$ and allowed to remain there for a period not less than 8 hours. Each specimen was then placed in a standard split ring Marshall apparatus and subjected to loading at a constant rate of 2 inches per minute deformation, until failure (maximum load reading) was obtained. Flow meter readings to the nearest 0.01 inch were recorded at the beginning and at the failure point for each test, and maximum load was recorded for each specimen tested.

RESULTS

Methods of Analysis

Three methods were utilized to determine shear strength parameters of each mixture at failure, i.e., the Mohr envelope, Bureau of Reclamation, and a modified stress path. For each specimen, recorded values of σ_1 and σ_3, the maximum and minimum principal stresses, were corrected for pore water pressure to determine the effective principal stresses, $\bar{\sigma}_1$ and $\bar{\sigma}_3$. The failure point was then defined as the point at which the effective stress ratio $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$ reached maximum value. The effective shear strength parameters, ϕ' and c', were then determined as described in the generalized methods sections noted below.

Mohr envelope

Figure 5 is a partial representation of the Mohr envelope analysis of the Garner sample treated with SS-1 emulsion.

Sowers and Sowers (29) explain that a German physicist, Otto Mohr, devised a graphical procedure for solving the equations for shear and normal stress on a plane perpendicular to one principal plane and making an angle, α, with the larger of the two other principal planes. On the abscissa, the values of normal stress are plotted, and on the ordinate, the values of shear stress are represented. Compressive (positive) normal stresses are plotted to the right of the zero normal stress axis, and tensile stresses are plotted to the left, while shear stresses may be plotted either upward or downward, as their sign has no meaning. On the representative figure, $\bar{\sigma}_1$, the maximum effective principal stress and, $\bar{\sigma}_3$, the minimum effective principal stress, both evaluated at specimen failure, are shown on the zero shear axis, since the shear stress on a principal plane is of zero value. The intermediate effective principal stress, $\overline{\sigma}_2$, is not represented on this plot, as it is assumed that $\overline{\sigma}_1$ and $\overline{\sigma}_3$ represent extreme conditions for the analysis, and $\overline{\sigma}_2$ and $\overline{\sigma}_3$ with a cylindrical specimen are assumed as equal. Through $\overline{\sigma}_1$, and $\overline{\sigma}_3$, a circle is drawn whose center is located at $\frac{\overline{\sigma}_1 + \overline{\sigma}_3}{2}$ on the abscissa, and whose radius is $\frac{\overline{\sigma}_1 - \overline{\sigma}_3}{2}$. A series of Mohr circles are plotted at the failure condition of maximum effective stress ratio for specimens tested at varying lateral pressures. On the representative figure, the circles were drawn for specimens tested at lateral pressures of 5, 15, 25, 40, 60, and 80 psi. A line drawn tangent to the circles is the envelope of failure defining the stress conditions on the failure plane of each specimen at each point of tangency. The angle of internal friction, ϕ', is the angle of the Mohr envelope measured from the horizontal, and c', cohesion, is the value determined on the ordinate at the point of envelope intercept.

Bureau of Reclamation

Figure 6 is a procedural representation of the Bureau of Reclamation method for determining the angle of internal friction and cohesion of the Garner sample treated with SS-1 emulsion. Values of ϕ' and c' are obtained by a statistical treatment, using the method of least squares and based on the Mohr envelope concept. The envelope of failure is assumed as a straight line when utilizing this procedure.

The method requires that at least three sets of values for $\overline{\sigma}_1$ and $\overline{\sigma}_3$, as determined at the maximum effective stress ratio of each specimen, are used in the analysis. The eight sets of data, represented in the figure, are the values determined for the treated Garner specimens. Thus, the values ϕ' and c' are the best that may be determined statistically for a straight Mohr envelope of failure, utilizing all available data.

Stress path

Stress path methods of analysis are intended to show the shear conditions in a specimen as it is being tested in the triaxial shear test. For the majority of tests, the lateral pressure, σ_3, is maintained constant while the axial load, σ_1, is increased to cause failure of the specimen.

The effective stresses are those which act on the individual soil particles. They may be determined by measuring the total stresses and applying a correction for measured values of pore water pressure. The effective maximum principal stress, $\overline{\sigma}_1$, and the effective minimum principal stress, $\overline{\sigma}_3$, may then be utilized to plot a modified stress path of the specimen as follows:

The effective stresses may be plotted on a shear strength diagram by the coordinates 0, $\frac{\overline{\sigma}_1 + \overline{\sigma}_3}{2}$ for center, and $\frac{\overline{\sigma}_1 - \overline{\sigma}_3}{2}$ for radius. If $p' = \frac{\overline{\sigma}_1 + \overline{\sigma}_3}{2}$ and $q' = \frac{\overline{\sigma}_1 - \overline{\sigma}_3}{2}$, a continuous plot of p' versus q' will indicate the state of stress during the triaxial shear test. Such a plot of p' versus q' describes a history of the stress change and produces a line, termed the "stress path". Figure 7 is a graphical representation of the stress path plot for an emulsion treated Garner specimen tested at 40 psi lateral pressure. Only three of the many possible Mohr circle representations are plotted on the graph to indicate the procedure utilized to determine the stress path line.

Figure 8 is a graphical presentation of the stress path plots for a series of Garner specimens tested at lateral pressures of 5, 15, 20, 25, 30, 40, 60, and 80 psi. The limiting line, or K_f line, which intercepts the maximum p'q' points for each test specimen, gives, by means of simple calculations, the values of the angle of internal friction, ϕ', and the cohesion, c'. These Mohr-Coulomb shear parameters may be determined by measuring the angle, α, that the line K_f makes with the horizontal and the Y intercept of the K_f line on the ordinate. The equations for ϕ' and c' are:

$$\tan \alpha = \sin \phi' \tag{1}$$

$$c' = Y \sec \phi'$$

In addition, the points at which the stress path plots of each specimen intercept the K_f line indicate a maximum stress condition.

The stress path method has a major advantage over the Mohr method in that it shows continuous change in stress to the point of failure. The concept is ideally suited for comparing effective stress paths of similar materials and gives an indication of the manner by which the material achieves full development of its maximum strength.

Comparison of methods of analysis

Tables 5, 6, and 7 present the values of ϕ' as determined by the three methods of analysis for all mixtures tested in this study. The values show little variation of angle of internal friction and cohesion for the individual mixes analyzed by each method. An apparent reason is that for each set of specimens, a straight line envelope of failure was developed.

It thus appears when many specimens are tested, and a straight line relationship is verified, any of the three methods of analyses will present reliable results. The Bureau of Reclamation method, however, is merely a least squares fit of the tangent line to the circles, drawn by the Mohr procedure. If a straight line relationship does exist, the BR method is the simplest for determing the values of ϕ' and c'.

The stress path method is the best graphical procedure for determining the values of ϕ' and c'. Utilizing the stress path method, the K_f line is drawn through a series of points, whereas the Mohr envelope of failure must be drawn tangent to each circle.

Thus the Mohr envelope method appears as the least precise analysis. A comparison of the data presented in Tables 5, 6, and 7 will indicate, however, that variations of ϕ' and c' are slight between each analytical method.

Use of the stress path method of analysis generally provides the most reliable sets of data as it is convenient to see a point which falls above, or below, a straight line connecting the majority of points. Such a point can be mentally disregarded as being non-representative of the material. The BR method, however, utilizes all of the data, including the non-representative point, and a statistically correct line is determined. Therefore, when increasing numbers of specimens are tested of each material, the BR method increases in reliability, providing that the straight line relationship is maintained.

The stress path method for determination of effective friction and cohesion was adopted for the bulk of shear strength parameter analyses contained in this study.

Maximum deviator stress analysis

Analysis of the major factors contributing to the possible mechanisms of specimen failure was conducted at the maximum effective deviator stress as defined by $\overline{\sigma}_1 - \overline{\sigma}_3$. The values of percent volume change, pore water pressure, and percent axial strain were determined for the treated and untreated specimens at the varying lateral pressures and at maximum deviator stress. Each of these factors was then plotted to provide comparative graphical data (Figures 9-15) of the various mixes.

In the preceding section of methods of analysis it will be noted that maximum effective stress ratio, $\frac{\overline{\sigma}_1 - \overline{\sigma}_3}{\overline{\sigma}_3}$, was assumed as the criterion of failure stress. Maximum effective deviator stress, $\overline{\sigma}_1 - \overline{\sigma}_3$, is the maximum stress condition applied on a cylindrical specimen subjected to axial compression with lateral support. Until the approximate date of Holtz's (17) report introducing the maximum effective stress ratio concept as the failure criterion for a triaxial shear test specimen, the maximum deviator stress condition was accepted as the failure criterion.

In the study reported herein, specimen stress values vary only slightly when they are evaluated at the maximum deviator stress condition and at the maximum stress ratio condition. Normally, quantitative values at the latter condition of stress are slightly less than those determined for the former condition.

Maximum stress ratio is currently accepted as a condition of specimen failure. The stress values determined at this condition, and at the maximum deviator stress condition, are in close proximity. Therefore, an investigation of the factors affecting failure of the bituminous treated stones herein, appears feasible at either condition. At maximum deviator stress, the factors are likely to have their greatest numerical values, and the exaggeration provides graphical data which are magnifications of the values occurring at the maximum effective stress ratio. The magnitude of the data, when presented graphically, provides an enlarged picture of the factors affecting failure, and thus provides improved comparative data.

For this study the factors of percent volume change, pore water pressure, and percent axial strain were analyzed at the maximum effective deviator stress condition.

Analysis of Data

Marshall stability test

The Marshall test method of mix design is primarily used to determine proper percentages of asphalt for maximum stability of asphalt paving mixtures. A curve, which resembles the optimum moisture-maximum dry density plot of soil mixtures, is obtained as percentages of asphalt are varied, test specimens produced and tested, and the data plotted. The "Marshall stability in pounds" (specimen failure load) is plotted as ordinate values, while "percent asphalt cement by weight of mix" is plotted as values on the abscissa.

For this study, utilizing the Marshall stability test, maximum loads and flow meter values were obtained for the AC treated specimens. The flow meter value indicates deformation of the test specimen as it is loaded to failure, and a value of "8" indicates a deformation of 0.08 inch. Stability criteria recommended by the Asphalt Institute (6) are basically for surface course hot-mix designs. No criteria have, as yet, been outlined for base course mixtures. However, the values indicated for surface course specimens provided indications of anticipated stability and flow meter values for the Marshall specimens tested during this study. For a surface course mixture designed for medium traffic, test specimens are to be compacted by application of 50 blows, from a standard compaction hammer, on both ends of the specimen. Five hundred pounds is the minimum allowable stability value for the mixtures, while the flow meter value is allowed to vary from a minimum of 8 to a maximum of 18. The test is normally conducted on specimens maintained at 140^{o}F (6).

For this study, a minimum of six Marshall test specimens of each mixture were molded in accordance with the methods of tests previously shown. However, no attempt was made to vary percentages of asphalt, as the Marshall tests were conducted primarily to indicate reliability and reproducibility of resulting density data for comparison with densities obtained by vibratory compaction of triaxial test specimens under similar conditions of gradation, percentage of asphalt, and temperature of mix at time of molding. Furthermore, the Marshall and triaxial specimens were tested at 100^{o}F temperature, in accordance with more recent Asphalt Institute criteria for test of bituminous base mixes. This was in contrast to the 140^{o}F noted above but is more realistic of temperatures acquired in highway base courses.

Tables 8, 9, 10, and 11 present the densities, Marshall stability loads and flow meter values obtained for the specimens tested by the Marshall procedure during this study.

Comparative densities of specimens compacted by the Marshall and vibratory methods are as follows:

1. The average dry density of the Marshall compaction Bedford total gradation specimens was 120.1 pcf. Of the seven specimens molded, the densities ranged from a low of 117.7 pcf to a high of 122.9 pcf. The maximum variation of any one specimen was 2.8 pcf. Comparative density of the vibratory compacted specimens was 119.6 $\pm$ 0.8 pcf*.

2. The average dry density of the Marshall compaction Bedford +200 dry-sieved specimens was 121.5 $\pm$ 1.7 pcf. For comparable vibratory compaction specimens, the average density was 119.9 $\pm$ 0.7 pcf; much more comparable to the total Bedford specimens.

3. For Marshall compaction Garner specimens, the average dry density was 142.6 $\pm$ 2.7 pcf for eight specimens, while the comparable average dry density of six vibratory specimens was 141.6 $\pm$ 1.1 pcf.

4. The average dry density of six Marshall compaction Gilmore specimens was 131.9 $\pm$ 1.9 pcf; for six vibratory specimens, 130.8 $\pm$ 1.2 pcf.

For all mixtures, the data thus indicated greater variations of densities for Marshall than for vibratory compaction specimens. The data, however, are somewhat misleading, since the vibratory specimens were molded to correspond to the average Marshall densities. The data does indicate though, a better reproducibility and reliability

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* The ($\pm$) value indicates the maximum density variation from the average for any test specimen.

of densification with vibratory than with Marshall compaction.

It appeared that variance of Marshall specimen densities was due primarily to soil particle degradation. As several specimens were removed from the mold, they were immediately discarded because the stone had been crushed during compaction, and the specimens were not suitable for comparative testing. Little or no particle degradation was noted for the vibratory specimens.

Though the majority of Marshall flow meter values fell within the range indicated as satisfactory for surface course mixtures (8-18), the Marshall stability values varied greatly within each mixture. The failure load for eight, total gradation Bedford specimens varied from a low of 142.4 pounds to a high of 175.9 pounds. For the Bedford +200 specimens, it varied from 159.9 to 208.3 pounds. For Garner and Gilmore specimens, values varied from lows of 185.4 and 141.3 pounds to highs of 274.7 and 325.6 pounds, respectively. In comparison, 500 pounds is the minimum Marshall stability load recommended for surface course mixtures.

The data presented appear to indicate relative stabilities of the four mixtures, however, the failure load of the mixtures varied considerably. For example, the failure loads for Gilmore specimens varied through a range of 184 pounds, as noted above. For the Marshall test, ASTM (5) recommends molding at least three specimens for each combination of aggregates and bitumen content. The failure loads recorded for six Gilmore specimens were 230.8, 176.6, 252.4, 164.3, 141.3, and 325.6 pounds. The densities of these specimens varied only ± 1.9 pcf from the average, and only one flow meter value (a value of seven for the specimen which failed at 252.4 pounds) did not fall within the range indicated for surface course mixtures. The failure load variations were not as great for all other mixtures as those presented above for the Gilmore specimens, however, there were substantial differences for each.

Bituminous-treated Bedford limestone

Because of its prominence in the state of Iowa, and because of its mixed service record, the Bedford stone was selected for the most extensive testing and analysis throughout this study.

Whole samples of Bedford stone were treated with two asphalt additives, 4.0% asphalt cement, and sufficient SS-1 emulsion to provide 3.0% asphalt residual. Asphalt contents were based on percentage of oven-dry aggregate.

A separate sample of Bedford stone was dry-sieved to remove all the fines passing the No. 200 U. S. Standard sieve, and was treated with 4.0% asphalt cement. The set of test specimens prepared from this mixture was analyzed to determine if the fines had a detrimental effect on the bituminous treatment.

Table 5 presents a tabular comparison of the densities, moisture contents, and shear strength parameters of the treated and untreated Bedford specimens with failure criteria based on the maximum effective stress ratio, $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$. For each mix, the reported density is the average of all specimens tested. The ($\pm$) value reported is the maximum variation from the average for any one of the test specimens.

Dry density for the emulsion treated specimens of 124.6 pcf was comparable to the value of 124.1 pcf obtained by standard Proctor compaction. The average moisture content of 8.0% was identical to the optimum moisture content indicated by Proctor test. The dry density of 119.6 pcf for the asphalt treated whole specimens was comparable to the average density obtained by the Marshall compaction of 120.1 pcf. For the Bedford specimens with the fines removed, the dry density of 119.9 pcf was comparable to the Marshall densities of 121.5 pcf. No moisture contents are recorded for specimens treated with asphalt cement, as they were in an oven-dry state at the time of compaction.

The average dry density of untreated specimens, 127.2 pcf, at optimum moisture content of 10.1%, was comparable to the standard Proctor values recorded in the materials section of this report, i.e., 127.4 pcf and 10.9%.

Densities of the bituminous treated Bedford specimens were less than those obtained for the untreated specimens and ranged up to an average reduction of 7.6 pcf for the AC treatment. Reduction of density was not as great with the emulsion treatment, being of the order of less than 3 pcf.

The value of 119.6 pcf for the AC treated whole sample and 119.9 pcf for the AC treated Bedford +200 specimens are comparable. Comparison of average densities for the AC treated specimens noted above, with the average density of the emulsion treated specimens, 124.6 pcf, and with that of the untreated specimens, 127.2 pcf, indicates a reduction of specimen density with increase of asphalt content. The soil particles are separated by the asphalt mastic in the specimens, and the asphalt binder (specific gravity = 1.0) increases the volume of the specimens but reduces the weight, thus reducing their densities.

The average moisture content of the emulsion treated specimens, 8.0%, is a reduction in comparison to that of the untreated specimens, 10.1%. The reduction of moisture contents to obtain the maximum densities of emulsion treated specimens was explained by Dunning and Turner (9). They presented a theory that the aqueous portion of the emulsion is "wetter" than the water, which is normally used for compaction, due to the presence of surface active emulsifying agents. The 2.1% reduction in average moisture content noted above, is comparable to the reduction of 1-3% observed by Dunning and Turner during their tests of similar materials.

Comparative data for the treated and untreated Bedford test specimens indicate a reduction of ϕ' and a corresponding increase of cohesion with the treated stone. The reduction is attributed to the lubrication and separation of soil particles by the asphaltic additive, while the increase in cohesion is derived from the binding characteristic. Stress path values of ϕ' and c' are respectively noted as 45.5° and 6.6 psi for the untreated specimens, 39.6° and 15.8 psi for the emulsion treated specimens, and 41.6° and 10.7 psi, and 39.0° and 15.4 psi for the asphalt cement treated whole and +200 sieve specimens, respectively. The shear parameter values derived by the Bureau of Reclamation and Mohr envelope methods, varied only slightly from those indicated above.

The slight variations of ϕ' and c' for the two series of asphalt cement treated specimens appear to indicate that the fines content has little effect on the bituminous treatment.

The slight variation of the shear strength parameters for specimens treated with 3% asphalt (emulsion treatment) and those treated with 4% asphalt cement indicates there is no significant reduction in specimen strength properties corresponding to a reduction of asphalt quantities.

A comparative analysis, based solely on shear strength parameters of treated and untreated Bedford specimens, provides data which indicate only a slight variation in stability of the stone treated with bituminous admixtures. The reductions in angles of internal friction for the treated specimens are at least partially counterbalanced by corresponding increases in cohesion. Therefore, an analysis of other factors affecting the failure of each test specimen was conducted at the maximum deviator stress condition.

Figures 9, 10, and 11 present graphical data of percent volume change, pore water pressure, and percent axial strain as determined at maximum deviator stress for all Bedford treated and untreated test specimens and as plotted against the varying lateral pressures.

Values of maximum deviator stress versus confining pressure indicate straight line relationships for the treated and untreated Bedford specimens though slope of the line for the untreated specimens is slightly greater than that of the treated. At low confining pressures, the maximum stress obtainable on the treated stone is greater than that on the untreated. For example, at 10 psi confining pressure, the maximum deviator stress of the untreated stone is approximately 75 psi, while that of the emulsion treated stone is 120 psi. At increased confining pressures, the deviator stress values of the untreated stone become greater than those of the treated. For example, at 70 psi confining pressure, the observed stress of the untreated stone was approximately 390 psi; for the emulsion treated stone, 285 psi. Similar straight line relationships were noted in comparing the untreated and asphalt cement treated specimens.

Due to the variations of stress increase for the treated and untreated specimen tests, there was a point for each comparison at which the maximum deviator stress for the two mixtures was identical. Up to this point, the treated specimens exhibited higher stress at failure than the untreated though after this point, the latter exhibited the greater stress values. This finding may be related to the findings of Endersby (10), who indicated similar comparisons for Mohr envelope analyses. He stated, that at a certain confining pressure, the Mohr failure envelope for the untreated specimens will cross that of treated specimens and there occurs a point of identical stress.

The slight slope reduction of the treated specimen failure envelope as compared with that of the untreated may be due to the lubricating and binding qualities of the bituminous additive.

The point of identical maximum effective deviator stress for the Bedford treated and untreated specimens varied with asphalt content and stone gradation. For the emulsion treated specimens, the point occurred at approximately 27 psi confining pressure, and for the asphalt cement treated whole and +200 sieve specimens it occurred at 15 and 27 psi confining pressures, respectively.

Comparative values of percent volume change for treated and untreated specimens indicated no significant variations at the maximum effective deviator stress. As lateral test pressures were increased, percent volume change decreased. At low lateral pressures, the volume change for both untreated and treated specimens tended to be positive, indicating a volume increase. As an example, at 10 psi lateral pressure, a +1.2% volume change was observed for the emulsion treated and untreated specimens. At higher lateral pressures, negative volume changes were observed for the test specimens, indicating a volume decrease. At an 80 psi confining pressure the approximate percent volume change for the emulsion treated and untreated specimens was -2.7%. Similar data were observed for the asphalt cement treated and untreated Bedford specimens.

Quantitative values of volume change at maximum deviator stress condition versus lateral confining pressures of the treated and untreated Bedford materials are approximately equal. This is indicative that the asphalt treatment does not improve the volume change characteristics of the untreated crushed stone.

Values of pore water pressure for treated and untreated Bedford stone indicated significant variations. Relatively straight line relationships were obtained for all mixtures. Pore pressures of the untreated stone increased, with increasing lateral confining pressures, whereas pore pressures of the treated stone remained nearly constant with increasing lateral pressure. At a confining pressure of 10 psi, the observed pore water pressures were -3.2 psi for the untreated specimens, -1.2 psi for the emulsion treated specimens, and -0.3 psi and -0.8 psi for the asphalt cement treated total gradation and +200 sieve specimens, respectively. At a confining pressure of 80 psi, the observed pore water pressures were +8.0 psi for the untreated specimens, +1.7 psi for the emulsion treated specimens, and +1.3 psi and +1.0 psi for the asphalt cement treated total gradation and

+200 sieve specimens, respectively.

The comparative pore water pressure values suggest the bituminous additive coats and waterproofs the soil particles. For any lateral pressure, only slightly negative or slightly positive pore water pressures were observed for the treated specimens. In contrast, the pore pressures of the untreated specimens varied substantially. At virtually all lateral pressures, the waterproofing effect of the asphalt significantly improved the pore pressure characteristics of the Bedford stone.

Capillary rise of moisture may be partially attributed to negative pore pressures. In effect, the negative pressure provides a suction which encourages the moisture rise. At low confining pressures, the bituminous treatment prevents the negative pore pressures associated with the untreated stone.

Positive pore pressures reduce the effective confining pressure, thus reducing the shear resistance of the specimens. At high lateral pressures, the bituminous additives prevented the highly positive pore pressures associated with the untreated stone.

At confining pressures of about 26 psi for emulsion treated specimens, 30 psi for asphalt cement total gradation specimens, and 27 psi for the AC treated +200 sieve specimens, pore pressures were identical for both the treated and untreated materials. This phenomenon may be explained by the relative change of pore water pressures, at varying lateral pressures, for all the mixtures.

Comparative values of percent axial strain, at conditions of failure, illustrate significant aspects of stability for the untreated and treated stones. Axial strain for the untreated specimens at all lateral pressures was approximately 6%, while the strains for the treated specimens varied from a low of 3.6% at 10 psi lateral pressure (asphalt cement treated total Bedford specimens), to a high of 11.2% at 80 psi lateral pressure (AC treated dry-sieved specimens). Similar values were observed for all treated specimens, the percent strains being low at low confining pressures and increasing corresponding to lateral pressure increases, indicating a positively sloped and relatively straight line relationship.

At low lateral pressures, maximum deviator stress values varied only slightly for the treated and the untreated specimens; for example, the maximum observed variation at 10 psi lateral pressure was 40 psi (the stress value was 120 psi for the emulsion treated specimens and 80 psi for the untreated specimens). Though the stress was greater for the treated stone, the corresponding strain was less than that of the untreated specimens. Similar results were apparent for all mix comparisons. The reduction of percent strain for the treated specimens was indicative of an increase in stability at low confining pressures.

As confining pressures were increased, maximum effective deviator stress values increased for all specimens. The strain of the treated specimens increased, but that of the untreated specimens remained nearly constant at the percentage indicated above. At 80 psi lateral pressure, the observed stress for the treated specimens was less than that of the untreated, while strain was significantly greater. Therefore at high lateral pressures, it appears that the asphalt mastic flows causing a relative decrease of stability in the treated specimens.

The combined stress-strain relationships indicate that bituminous admixtures improve stability of the Bedford materials subjected to low confining pressures, (those less than an approximate 30 psi value as indicated on the figures). However, the additive treatment actually decreases stability at higher confining pressures. This latter phenomenon may be attributed to a flow of the asphalt mastic within the specimens which prevents complete grain to grain contact of soil particles.

Further indications of this flow phenomenon will be discussed in the section of this report titled "Volume change phenomenon".

Bituminous-treated Garner dolomitic limestone

Whole Garner samples were treated with 4.0% asphalt cement and sufficient asphalt emulsion to provide a 3.0% asphalt residual, as based on the weight of oven-dry aggregate.

Table 6 presents tabular comparisons of densities, moisture contents, and shear strength parameters for the treated and untreated stone. For this portion of the study, failure criteria were based on the maximum effective stress ratio, $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$, of the test specimens.

For each mix, the reported density is the average of all specimens tested. The ($\pm$) value signifies the maximum variation from the average for any of the test specimens. The dry density of 142.9 pcf at a moisture content of 5.4% for the emulsion treated specimens is comparable to the density of 143.4 pcf at a 5.8% moisture content achieved by Proctor compaction. The average dry density of 141.6 pcf for the asphalt cement treated specimens is comparable to the Marshall density of 142.6 pcf. No moisture content is reported for the latter treatment since specimens were in an oven-dry state at time of compaction. The dry density of 145.4 pcf and 6.8% moisture content for untreated specimens is comparable to the standard Proctor values recorded in the materials section of this report, (140.5 pcf and 7.6%).

As noted with the Bedford specimens, the densities of treated Garner specimens were less than the average density of the untreated stone. Furthermore, the densities of the specimens treated with 4.0% additive exhibited greater variations from the densities of the untreated specimens than those treated with 3.0% asphalt. The average density reduction of the former was 3.8 pcf; of the latter, 2.5 pcf. The variations appear to indicate that the asphalt increases specimen volume by separating the soil grains, but decreases their weight, thus contributing to the density reduction. The fact that the density variation was less for specimens treated with 3.0% additive than that of the specimens treated with 4.0% additive further verifies this theory. The average moisture content of 5.4% for emulsion treated specimens is less than the 6.8% of the untreated specimens. The phenomenon of moisture reduction was explained by Dunning and Turner (9) and has been presented in preceding sections. The 1.4% moisture content reduction noted above is comparable to the reduction of 1-3% observed by Dunning and Turner during their investigation of similar materials.

Comparison of the treated and untreated specimens indicate reductions of angles of internal friction with corresponding increases of cohesion for the treated stone. Reduction of ϕ' may be attributed to lubrication of soil particles by bituminous admixtures, while the increase of c' is due to the binding quality of the asphalt products.

The shear strength parameters, ϕ' and c', derived by the stress path method for the treated and untreated stone are as follows:

1. For the untreated stone, $\phi' = 49.8^{o}$ and c' = 13.9 psi.
2. For the emulsion treated specimens, $\phi' = 45.5^{o}$ and c' = 16.9 psi.
3. For the asphalt cement treated specimens, $\phi' = 42.9^{o}$ and c' = 16.0 psi.

Parameters derived by the Mohr envelope and Bureau of Reclamation methods were similar to those noted above.

Indicated is a reduction of ϕ' with corresponding increase in c' for the specimens treated with 4.0% additives as compared to the specimens treated with 3.0% additive. The reduction of ϕ' is at least partially counterbalanced by an increase in cohesion, so that 3.0% and 4.0% additives provide mixtures of nearly identical shear strength characteristics.

A comparison, based solely on the shear strength parameters, of treated and untreated specimens provides data which indicate only slight variations in stability. Reduction of angles of internal friction for the treated specimens are at least partially counterbalanced by the corresponding cohesion increases. Therefore, an analysis of other factors affecting the failure of each test specimen was conducted at the maximum deviator stress condition.

Figures 12 and 13 present graphical comparisons of percent volume change, pore water pressure, and percent axial strain as determined at maximum deviator stress for the treated and untreated Garner test specimens and as plotted against the varying lateral pressures.

Values of maximum deviator stress versus confining pressure indicate straight line relationships for the treated and untreated stone. As noted for the Bedford specimens, the slope of the line for the untreated Garner stone is somewhat greater than that of the treated specimens. At low confining pressure, the maximum effective deviator stress obtainable for the treated stone is greater than that of the untreated. For example, at 10 psi confining pressure, the stress on the untreated specimens was approximately 110 psi, while the stress on the emulsion treated specimens was 145 psi and on the AC treated specimens, 120 psi. As lateral pressures were increased, the stress recorded for both treated and untreated specimens increased, though the relative increase was less for the treated specimens. At high lateral pressures, the stress on the untreated stone was greater than that on the treated due to the relative change indicated. For example, at 70 psi lateral pressure, the stress on the untreated specimens was about 530 psi, while the stress on the emulsion treated specimens was about 410 psi, and on the AC treated specimens, about 385 psi.

Due to the noted variations of stress increase for the treated and untreated specimens, there occurred a point for both comparisons at which the maximum deviator stress for the two mixtures were identical. Up to this point, the treated specimens exhibited higher stress at failure than the untreated, and after this point the untreated specimens exhibited the higher stress values. This occurrence may be related to the findings of Endersby (10), who indicated similar occurrences for the Mohr failure envelopes for treated and untreated specimens. The point of identical stress for the Garner emulsion treated and untreated specimens occurred at about 28 psi lateral pressure, and for the untreated and AC treated specimens it occurred at approximately 14 psi.

Comparative values of percent volume change for treated and untreated specimens indicated no significant variation at maximum deviator stress conditions though at each lateral pressure the volume change was about 0.5-0.7% greater for the untreated than for the treated specimens. As lateral pressures were increased, the percent volume change for all mixtures decreased from positive to negative values, i.e., from volume increase to volume decrease. At the 10 psi confining pressure, the percent volume change reported for the untreated stone was +0.9%, while for the emulsion treated and AC treated stones, it was +0.4 and +0.5%, respectively. At 80 psi pressure, the untreated stone percentage volume change was a negative 1.2%, and for the emulsion and AC treated specimens, the volume change was -2.0 and -1.7%, respectively.

Quantitative values of volume change at maximum deviator stress condition versus lateral confining pressure of treated and untreated specimens are approximately equal. This is indicative that the asphalt treatment does not improve the volume change characteristics of the untreated crushed stone.

Values of pore water pressure for the treated and untreated Garner stone indicated relatively straight line relationships for all mixtures. Pore pressures of the untreated stone increased, with increasing lateral confining pressure, whereas pore pressures of the treated stone remained nearly constant with increasing lateral pressure. By comparison of Figures 12 and 13, with Figures 9, 10, and 11 of the Bedford stone, it may be observed that the pore pressures of the two untreated stones are significantly different; the Bedford being much greater than the Garner.

At 10 psi confining pressure, the pore water pressure of the untreated Garner stone was approximately -3 psi, while that of the emulsion treated stone was -1 psi, and that of the AC treated stone was -0.6 psi. As lateral pressures increased, the pore water pressures of the treated stone approached zero, while those of the untreated stone increased to low positive values. In the vicinity of 60 psi lateral pressure, pore pressures of the treated and untreated specimens were about equal.

At low confining pressures, the bituminous admixtures reduced the negative pore pressures associated with the untreated stone, thus improving its quality, but at higher lateral pressures there was relatively little change in pore pressures for the mixtures. Negative pore pressures encourage capillary moisture rise in base materials. The waterproofing of the stone by the asphalt additives sealed the pores preventing significant negative pore pressures at low confining pressures, thus improving the general water stability characteristics.

Comparative values of percent axial strain illustrate significant aspects of stability for the treated and untreated Garner stone. Strain at failure of the untreated stone remained relatively constant at approximately 4%, irregardless of the confining pressure. As confining pressure varied from 10 to 80 psi, the corresponding strains varied from 3.1 to 7.4% for the emulsion treatments, and from 1.3 to 2.9% for the AC treated specimens.

At low lateral pressures, maximum deviator stress values varied only slightly for the treated and untreated specimens. For example, the maximum observed variation at 10 psi lateral pressure was approximately 20 psi (the stress value was 130 psi for AC treated specimens and 110 psi for the untreated). Though stress of the AC treated specimens was greater than that of the untreated, the stress of emulsion treated specimens was approximately equal to that of the untreated, the corresponding strains of treated specimens were less than the strain of the untreated stone. Similar occurrences were reported for Bedford treated and untreated specimens at low confining pressures.

As confining pressures were increased, the maximum effective deviator stress increased for all specimens. Strain of treated specimens increased but that of the untreated specimens remained nearly constant at the percentage indicated above.

At higher confining pressures the observed stresses for treated specimens were less than the stress of the untreated. Corresponding strain of the emulsion treated specimens was greater than that of the untreated stone, and the strain of the AC treated specimens was slightly less than that of the untreated stone. Therefore, at high lateral pressures, it appears there is a definite reduction of stability of the emulsion treated, relative to the untreated, specimens. There also appears a reduction of stability of the AC treated specimens which is not readily evident on Figure 13. At 80 psi confining pressure, the maximum effective deviator stress reported for untreated specimens was 610 psi, and that for AC treated specimens, 425 psi. This 185 psi stress variation was significant. The corresponding percentage strains reported were approximately 4% for untreated specimens, and 3% for the AC treated stone. The relative stress-strain variations of the AC treated specimens, indicate a similar reduction in stability as that referenced above for the emulsion treated specimens. The additive treatment actually decreased stability at higher confining pressures. This phenomenon may be attributed to a flow of the asphalt mastic within the specimens which prevents complete grain to grain contact of soil particles.

Further indications of this flow phenomenon will be discussed in the section of this report titled "Volume change phenomenon".

Bituminous-treated Gilmore limestone

Whole Gilmore samples were treated with 4.0% asphalt cement, and sufficient SS-1 emulsified asphalt to provide 3.0% residual as based on the weight of oven-dry aggregate.

Table 7 presents tabular comparisons of densities, moisture contents, and shear strength parameters for the treated and untreated stone. For this portion of the study, failure criteria was based on the maximum effective stress ratio, $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$, for the test specimens. For each mix, the reported value of density is the average for all specimens tested. The ($\pm$) value signifies the maximum variation of density for any test specimen.

Dry density of 132.8 pcf at a moisture content of 5.0% for the emulsion treated specimens is comparable to the density of 130.5 pcf at a 5.2% moisture content achieved by standard Proctor compaction. The average dry density of 130.8 pcf for the asphalt cement treated specimens is comparable to the Marshall density of 131.9 pcf. No moisture content is recorded for the latter treatment since specimens were in an oven-dry state at time of compaction. Dry density of 133.2

pcf at a moisture content of 6.9% for the untreated specimens is relatively comparable to the values reported in the materials section of this report (130.8 pcf and 9.4%).

As was previously noted for Bedford and Garner specimens, densities of treated Gilmore specimens were less than the average density of the untreated. Furthermore, the samples treated with 4.0% AC exhibited greater variation from the untreated specimen density than did the specimens treated with 3.0% residual additive. The variations appear to indicate that the asphalt increases specimen volumes by separating soil grains, but decreases the specimen weights, thus reducing densities. The data indicating excess density reduction for samples treated with 4.0%, in contrast to those treated with 3.0% additive, further verifies this theory.

The average moisture content of 5.0% for emulsion treated specimens is less than the 6.9% of untreated specimens. The moisture content reduction phenomenon was explained by Dunning and Turner (9) as reported in preceding sections. The 1.9% moisture content reduction indicated above is comparable to the 1-3% reductions noted by Dunning and Turner during their study of similar materials.

Comparative data for treated and untreated specimens indicate reductions of angles of internal friction with corresponding increases of cohesion for the treated stone. The reduction of ϕ' is probably due to lubrication of soil particles by the bituminous admixture, while the increase in cohesion is derived from the binding quality of the asphalt. Shear strength parameters for treated and untreated specimens, derived by the stress path method are as follows:

1. For untreated specimens, $\phi' = 46.2^{o}$ and $c' = 13.4$ psi.

2. For emulsion treated specimens, $\phi' = 40.6^{o}$ and $c' = 19.4$ psi.

3. For AC treated specimens, $\phi' = 42.4^{o}$ and $c' = 14.4$ psi.

Parameters derived by the graphical Mohr envelope and Bureau of Reclamation methods varied only slightly from those noted above.

Values of ϕ' and c' for treated specimens indicates there is little change in shearing strength derived from using 3.0% or 4.0% additives. A comparative analysis, based solely on shear strength parameters of treated and untreated specimens, provides data which indicate only slight variations in strength. Strength reductions associated with the reductions of angles of internal friction for the treated specimens are at least partially counterbalanced by increased cohesion. Therefore, an analysis of other factors affecting the failure of each test specimen was conducted at the maximum deviator stress condition.

Figures 14 and 15 present graphical data for comparative analyses of percent volume change, pore water pressure, and percent axial strain as determined at maximum deviator stress for all Gilmore treated and untreated test specimens, and as plotted against the varying lateral pressures.

Values of maximum deviator stress versus confining pressure indicate straight line relationships for treated and untreated specimens. As reported for the Garner and Bedford specimens, the slope of the line for the untreated Gilmore specimens was greater than that of the treated. At low confining pressures, the maximum effective deviator stress obtainable was greater for the treated than the untreated stone. For example, at 10 psi lateral pressure, the stress on untreated specimens was 110 psi, while stress on the emulsion treated stone was 130 psi, and on the AC treated stone it was just slightly higher than the value of the untreated. As lateral pressures increased, the stress values for both treated and untreated specimens increased, though values for the latter increased more rapidly than those of the former. At high confining pressures, the observed untreated specimen stress was greater than the stress on the treated stone. At 80 psi lateral pressure, stress on the untreated specimens was 490 psi, while stress on the emulsion treated and AC treated specimens was about 390 psi.

Due to the noted variations of stress increase for the treated and untreated specimens, there occurred a point for both the AC and emulsion-untreated stone comparisons, at which the stress at failure for the treated and untreated specimens was identical. Up to this point, the treated specimens exhibited higher stress at failure than the untreated. Following this point, the untreated specimens exhibited higher obtainable stress conditions. A similar occurrence was noted by Endersby (10) in his study of Mohr envelope analyses and is referenced earlier in this report. The point of identical stress for the Gilmore emulsion treated and untreated specimens occurred at about 23 psi confining pressure and for the untreated and AC treated specimens it occurred at approximately 11 psi.

Values of percent volume change for treated and untreated specimens indicated no significant variations at maximum deviator stress. As lateral pressures were increased, percent volume changes for all mixtures varied from positive to negative values, i.e., from volume increase to volume decrease, and were similar for all materials. At

10 psi confining pressure, percent volume change for the untreated stone was about +0.9%, while for the emulsion treated and AC treated stones, it was about +0.5 and +0.2%, respectively. At 80 psi lateral pressure, the untreated stone percentage volume change was a negative 1.0%, and for emulsion and AC treated specimens, the volume change was -1.4 and -0.7%, respectively.

Variation of quantitative values of volume change at maximum deviator stress condition versus lateral confining pressure between treated and untreated specimens was relatively small. This is indicative that the asphalt treatment does not significantly improve the volume change characteristic of the untreated crushed stone. A slight, but relatively insignificant, improvement is evident at all confining pressures with the emulsion treatment but only at lower lateral pressures with AC treatment.

Comparisons of the pore water pressure for the treated and untreated specimens indicated only slight variations when related to the values indicated for the Bedford stone. At low confining pressure the pore pressures of all mixtures were nearly identical. At 10 psi confining pressure, for example, the pore pressure of the untreated stone was -1.6 psi, of the emulsion treated stone approximately -0.8 psi, and of the AC treated stone, about -0.8 psi. As lateral pressures increased, pore pressures of the treated stone increased in an approximate straight line relationship, but never increased above about 1.0 psi (at the 80 psi lateral pressure, the observed pore pressure for the AC treated specimens was +0.0 psi, and for the emulsion treated specimens, +0.7 psi). Figures 14 and 15 indicate that the untreated specimen pore pressures also increased, but the plotted values presented a curve which was concave downward though in general increasing in value with additional confinement. For example, at 80 psi lateral pressure, the pore pressure was approximately 2 psi.

Several observations may be made from the pore pressure versus lateral pressure data. (1) The bituminous admixture waterproofs the soil particles since the data indicate that pressures varied from a maximum low of about -1.0 psi to a maximum high of _1.2 psi for the treated stone. In contrast, the pore pressures of the untreated stone varied from -1.6 psi to +1.8 psi. (2) The relatively low pore pressures evidenced for the untreated stone indicate that it is not as susceptable to pore water pressures as the Bedford. (3) Waterproofing of the stone by asphalt, prevents development of significant pore pressures thus improving the stones general water susceptibility characteristic.

Stability of treated and untreated specimens was also indicated by the graph of percent axial strain at maximum deviator stress versus lateral pressure for all Gilmore specimens. For untreated specimens the percent axial strain remained nearly constant at about 5.6%. However, as confining pressure varied from 10 to 80 psi, corresponding strains varied from about 3.0 to 10.0% for the emulsion treated stone, and from about 3.0 to 7.6% for the AC treated stone.

At low lateral pressures, maximum deviator stress conditions were similar for treated and untreated specimens, while at increased lateral pressures they varied significantly, being of the magnitude of 90 psi for emulsion treated specimens and 80 psi for AC treated specimens measured at a confining pressure of 80 psi.

At low confining pressures, the low value of percent strain associated with treated specimens is an indication of increase in stability, in comparison with the untreated specimens. At the higher lateral pressures, the untreated specimens exhibited less axial strain indicating that at these confining pressures, asphalt treatment is detrimental to stability.

Reduction of treated specimen stability at high confining pressures may be related to a flow characteristic of the asphalt mastic which prevents complete contact of soil grains and reduces cohesion. Further indications of the flow phenomenon will be discussed in the next section of this report.

Volume change phenomenon

The data obtained during the triaxial shear test were written in computer form for a specially prepared IBM 7074 program. The computer program was then used in a 1627 plotter which graphed effective stress ratio, volume change, and pore pressure, versus percent strain for each specimen tested. Figure 16 is representative of the computer plots for all asphalt mixes in this study.

In the literature search, it appeared that similar data for triaxial tests indicated that maximum negative volume occurred at, or very near, the failure point indicated by the maximum effective stress ratio. Minimum volume change correlation with $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$ was indicated for all materials tested irregardless of additive or treatment.

In this study, a significant phenomenon, which is in contrast to the findings noted above, was evidenced by means of the computer graphs, similar to Figure 16. As shown thereon, the minimum volume occurred sometime after the point of maximum stress ratio was achieved for the majority of all bituminous-treated test specimens. The grahpical representations and the itemized data indicated that minimum volume occurred at a percent strain much greater than that achieved at maximum stress ratio.

If the above occurrence has been realized by other investigators, the authors of this report were unable to find it in print. Therefore, it is felt that a possible cause of the observed occurrence is that the asphalt cement between the specimen's particles, flows during triaxial testing. Failure of the specimen may be due to a shearing of the soil grains on the weakest plane. Soil grains within the remainder of the specimen may not have come into complete contact at the failure condition, due to the asphalt film causing soil particle separation. As axial strain is increased, after stress failure occurs, asphalt cement continues to flow until all soil grains have made contact. Only then, does the volume reach its minimum value and begin to increase.

Comparison of computer plots for specimens treated with 4% additive and those treated with 3% additive indicated that minimum volume for the latter specimens occurred nearer the maximum effective stress ratio condition than had the minimum volume for the former. The variance may be attributed to two major factors. (1) Increase of asphalt content of specimens treated with 4% additive increases the asphalt film thickness between soil grains, contributing to increased lag in attaining soil grain contact and minimum volume. (2) Asphalt cement treated specimens were tested at $100^{\circ}F$ temperature, while emulsion treated specimens were at room temperature at the time of testing.

The slight increase of temperature would increase the flow characteristic of the asphalt admixture, contributing to the occurrence of the observed phenomenon.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

The following is a composite summary based on the data, test results, and published literature observed during this study.

Through the literature review, and during the analysis of all available data, the importance of the triaxial shear test became obvious. This test provides the best single laboratory indication of failure criteria and contributing factors of failure for bituminous treated and untreated base course mixtures.

A comparison of the densities, moisture contents, and shear strength parameters of the treated and untreated specimens indicated the following:

1. Densities of the untreated specimens were greater than densities of the treated specimens. This occurrence is probably due to the separation of soil particles with asphalt cement which has a specific gravity of nearly 1.0. Volume of the treated specimens is increased while the weights are decreased, contributing to the density reduction.
2. Moisture contents at maximum dry density of the untreated stone were greater than those of the emulsion treated. Dunning and Turner (9) have explained this phenomenon as being attributable to the "wetter" aqueous solution caused by surface active emulsifying agents in the emulsion liquid.
3. Densities of treated specimens compacted by the vibratory method are comparable to densities achieved by Marshall and Proctor compaction methods.

4. Shear strength parameters for specimens treated with 3.0% and 4.0% asphalt additive exhibited only slight variations. The data appear to indicate a reduction in asphalt content from 4.0% to 3.0% will not greatly affect the general shear and strength stability of the stone.

5. Straight line Mohr envelopes of failure were observed for all of the treated stones. The effective angles of internal friction were somewhat less than those of the untreated stone. Correspondingly, the effective values of cohesion for the untreated stones were less than those for the treated. This phenomenon is probably caused by the lubrication and binding qualities of the bituminous additives.

Values of ϕ' and c' for treated and untreated specimens did not vary significantly. Thus it was impossible to determine complete success of failure of the additive treatment based solely on shear strength parameters. A comparative analysis of the percent change in volume, pore water pressure, and percent axial strain was conducted for the treated and untreated stones at the maximum deviator stress condition of failure. A composite comparison is as follows:

1. Maximum deviator stress at various lateral pressures indicated straight line positively sloped relations for treated and untreated stones. For each plot, the slopes of the line for the treated stone were greater than those for the untreated. At low confining pressures, maximum deviator stress of the treated stone was greater than that of the untreated. As lateral pressures were increased, corresponding increases in stress of the mixtures reached

a point of equal value. After this point, stress of untreated specimens at failure was greater than that of the treated.

2. Percent volume change at various lateral pressures for treated and untreated specimens, indicated that bituminous admixtures did little to improve volume change characteristics of the untreated stones. As lateral pressures were increased, percent volume change for all mixtures decreased from positive to negative values, i.e., from volume increase to volume decrease at failure. For the Bedford stone, percent volume change for untreated and treated specimens was approximately equal at all confining pressures. For the Garner stone, there appeared a 0.5-0.7% difference in present volume change for treated and untreated specimens, though both had nearly identical slopes, again indicating little or no improvement of volume change characteristics associated with the bituminous admixtures. Slight variations existed for Gilmore treated and untreated specimens. Relatively insignificant improvements of untreated stone volume change characteristics were noted for AC and emulsion treated Gilmore specimens tested at low confining pressures. At higher pressures, some improvement was evidenced for emulsion treated Gilmore specimens only.

3. Figure 17 illustrates that the Bedford stone particles were not as effectively covered by asphalt additives as were the

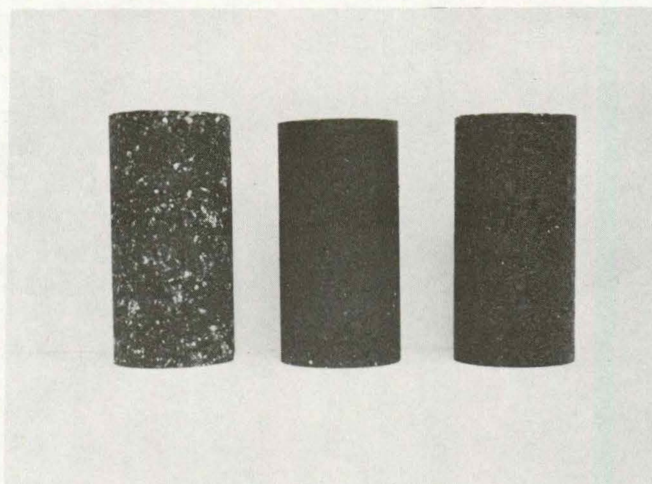

Figure 17. Bedford, Garner, and Gilmore tri-
axial test specimens illustrating
incomplete asphalt cement coverage
of Bedford aggregate particles

Garner and Gilmore aggregates. However, representative pore water pressure values indicated that bituminous treatment notably improved only the Bedford sample by effectively waterproofing and sealing the pores of the stones. Waterproofing reduced excessively high and low pore pressures associated with untreated Bedford specimens, at corresponding high and low lateral pressures. By reducing high pore pressures, the admixtures, in effect, increased the shear strength of the stone, and by reducing negative pore pressures, the additives effectively reduced suctions which contribute to capillary moisture rise within the base material. Gilmore and Garner stones, apparently being less porous than Bedford, exhibited only slight improvement when treated with bituminous additives.

4. Percent axial strain at maximum deviator stress versus changing lateral pressure indicates base course stability. At low confining pressures, bituminous treatments increase the stability of the stone by reducing the strain characteristic. Reduction of strain is probably effective in reducing deformation and rutting of highway base course mixtures, so long as low lateral pressures can be observed. At higher lateral pressures, the treated stone appears less stable than the untreated. This phenomenon is probably due to the flow characteristic exhibited with the bituminous additive.

For the majority of triaxial shear tests conducted on the treated stones, minimum volume occurred sometime after the achievement of maximum effective stress ratio. Literature which references the subject, indicates that minimum volume normally occurs at, or near, the point of maximum effective stress ratio. It is theorized herein, that the observed phenomenon is caused by the flow characteristics of the bituminous mastic. Though failure may occur due to shear of the stone along the weakest plane, not all of the soil grains within the specimen have achieved complete contact due to the asphaltic film separation. As the axial load is increased beyond the failure point, the additive will flow until the soil grains are in complete contact. Then as soil particles re-orient, the volume begins to increase.

The observed minimum volume lag was greater for specimens treated with 4% than for those treated with 3% additive. The greater lag is apparently due to increased asphalt content, and somewhat higher test temperatures associated with the asphalt cement treated specimens.

The following are recommendations based on the tests performed, and the analyses presented:

1. Though at low confining pressures there appears an increase of stability for the stones treated with bituminous admixtures, the additive should be used only with the Bedford stone to reduce negative pore pressures and, in effect, reduce capillary moisture rise within the base course mixture. Asphalt contents of one or two percent might be sufficient for achieving this waterproofing objective, while stability of the mixture would then be dependent on the mechanical stabilization. Baskin and McLeod (7) have discussed such a concept.

2. In accordance with the same procedures used in this study, tests of specimens treated with asphalt cement contents of one or

two percent should be conducted to determine variations in stability corresponding to reductions of asphalt contents.

3. Specimens treated with 1 or 2% asphalt cement should be subjected to wet-dry, and freeze-thaw tests to determine the possibility of their improvement as compared to untreated specimens subjected to similar tests. Such tests would assist in analyzing the effectiveness of the asphalt as a waterproofing agent.

4. Tests should be conducted to further analyze the volume change lag phenomenon. A repetitive axial load triaxial shear test should be utilized early in the investigation. As it is anticipated such a test may indicate a decrease of the volume change lag through cycling of load.

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APPENDIX

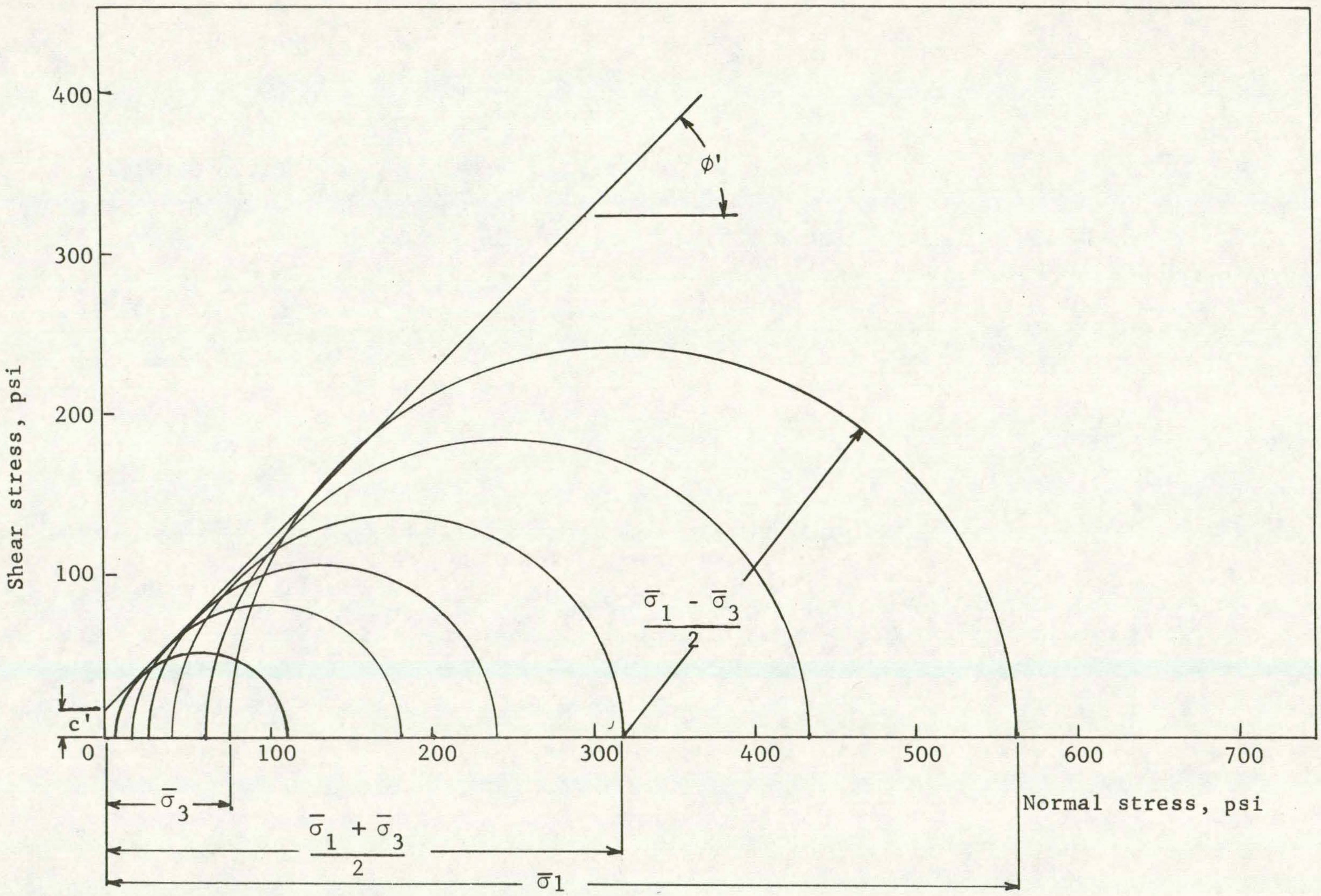


Figure 5. Illustrative example of the Mohr envelope; emulsion treated Garner specimens tested at lateral pressures of 5, 15, 25, 40, 60, and 80 psi

Specimen No.	Test No. and σ_3	$\overline{\sigma}_1$	$(\overline{\sigma}_1)^2$	$\overline{\sigma}_3$	$(\overline{\sigma}_3)^2$
139	1- 5	111.4	12409.9	5.2	27.0
138	2-15	184.7	34114.0	15.1	228.0
238	3-25	240.8	57984.6	25.1	630.0
239	4-40	215.4	46397.1	20.9	436.8
339	5-30	244.8	59927.0	30.3	918.0
439	6-40	323.4	104587.5	40.1	1608.0
632	7-60	436.1	190183.2	60.0	3600.0
839	8-80	564.2	318321.6	79.7	6352.0
Σ	n=8	2,320.8 (1)	823,924.9 (2)	276.4 (3)	13,799.8 (4)

$[n \times (2)] - (1)^2$

$= [8 \times 823,924.9] - (2,320.8)^2$

$= 1,205,286.6$ (9)

$[n \times (4)] - (3)^2$

$= [8 \times 13,799.8] - (276.4)^2$

$= 34,001.5$ (10)

$$A^2 = \frac{(9)}{(10)} = \frac{1,205,286.6}{34,001.5} = 35.4$$

$$A = 5.950, \ \sqrt{A} = 2.439, \ 2\sqrt{A} = 4.878$$

$$\text{Cohesion} = \frac{(1)-[A \times (3)]}{n2\sqrt{A}} = \frac{2320.8 - [5.950 \times 276.4]}{8(4.878)}$$

$$= 17.3 \text{ psi} \qquad c'$$

$$\text{Tan } \phi' = \frac{A - 1}{2\sqrt{A}} \quad \frac{5.950 - 1.0}{4.878} = 1.014 \qquad \tan \phi'$$

$$\phi' = 45.4^{\circ} \qquad \phi'$$

Figure 6. Bureau of Reclamation procedure for determining ϕ' and c' of the emulsion treated Garner sample.

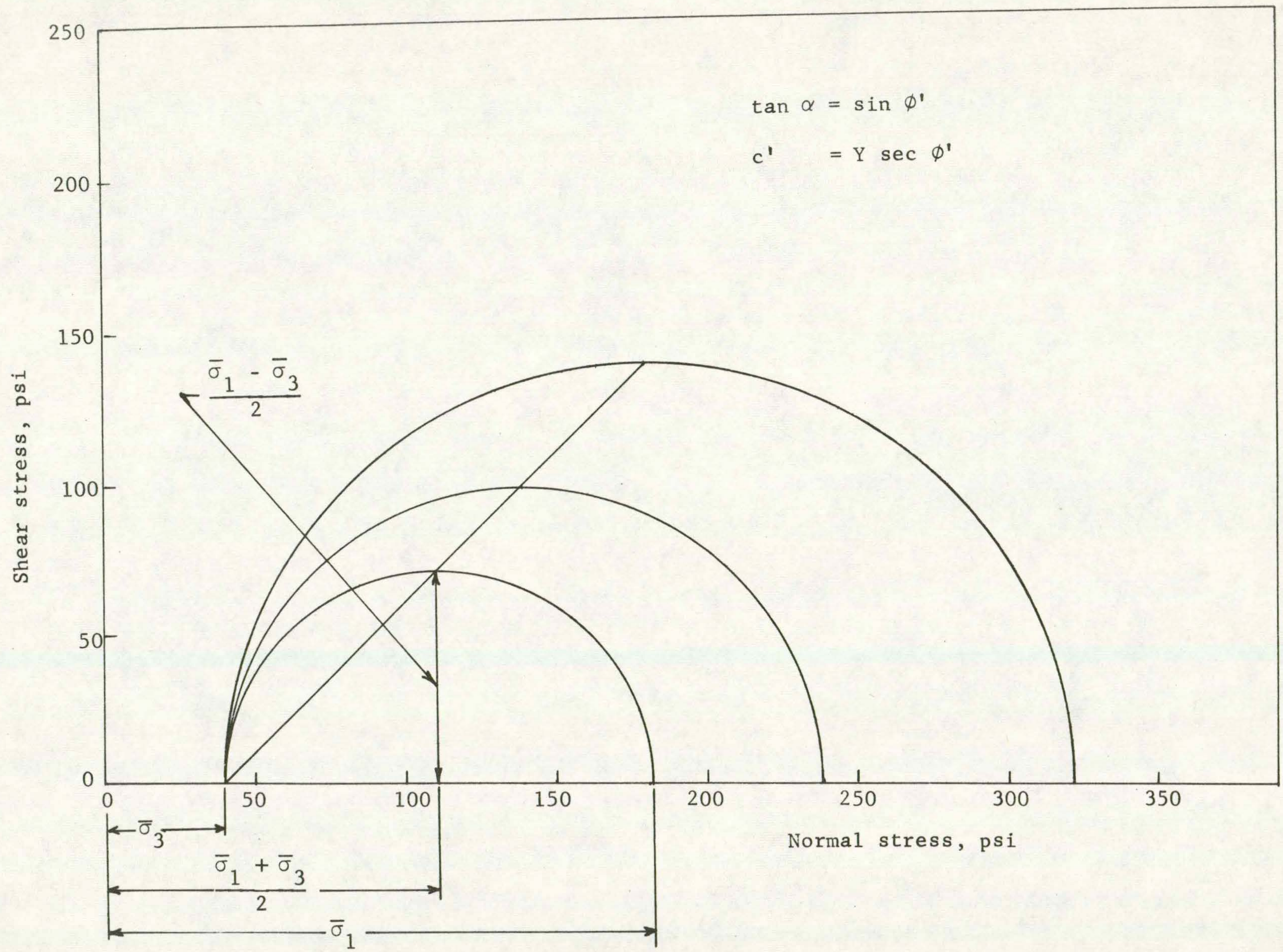


Figure 7. Stress path representation by Mohr circle maximum ordinate for Garner sample treated with SS-1 emulsion and tested at 40 psi lateral pressure

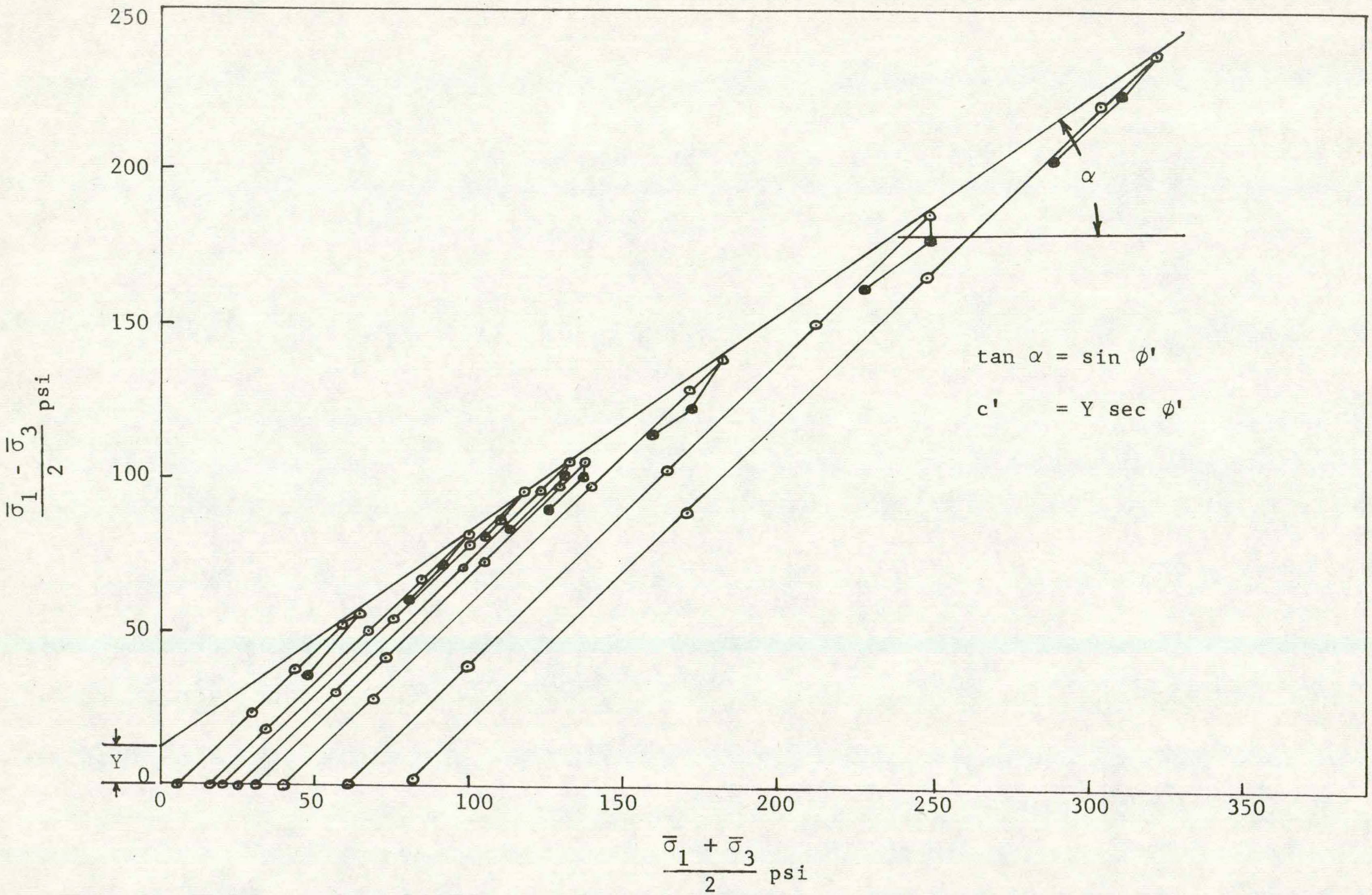
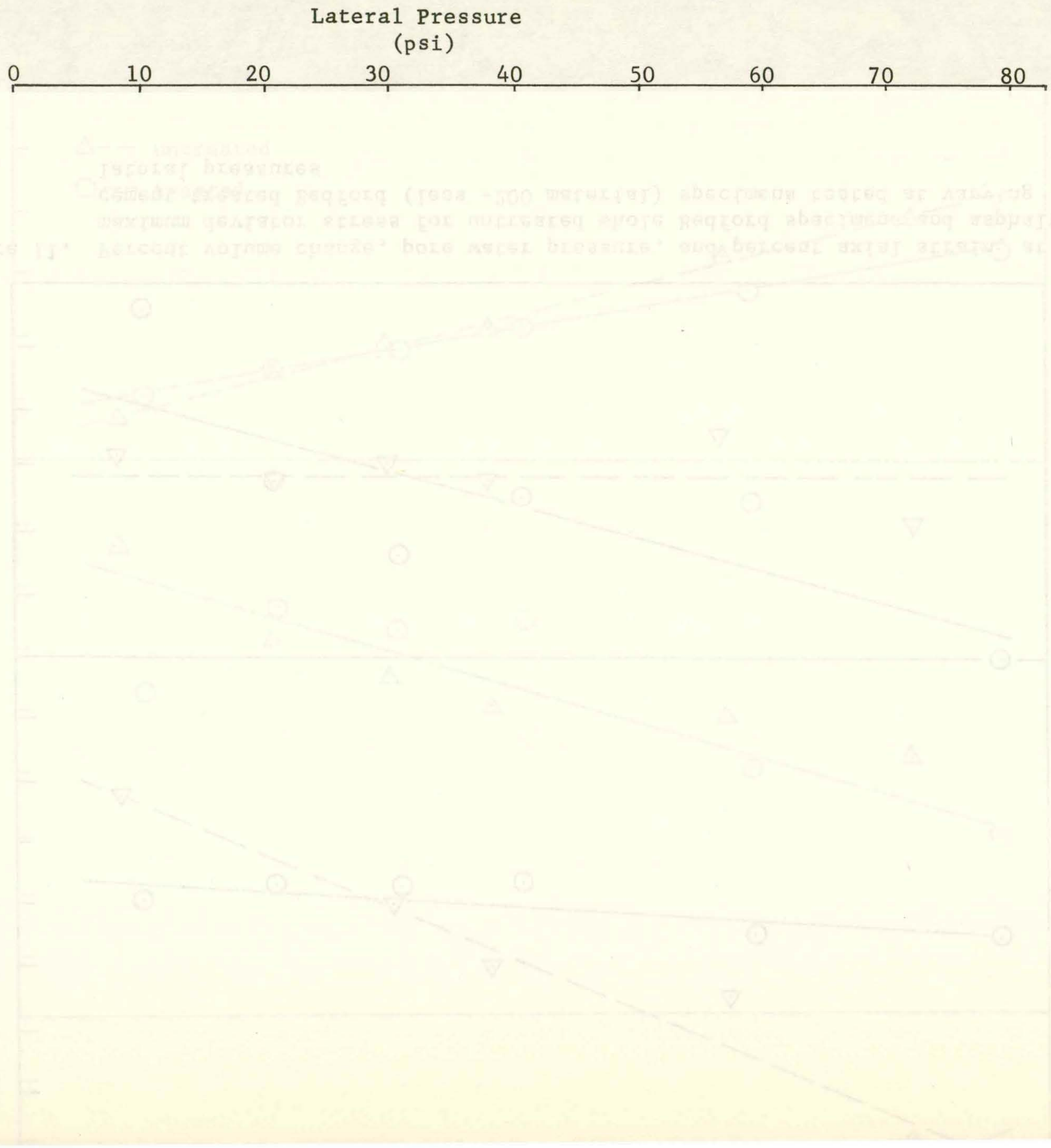


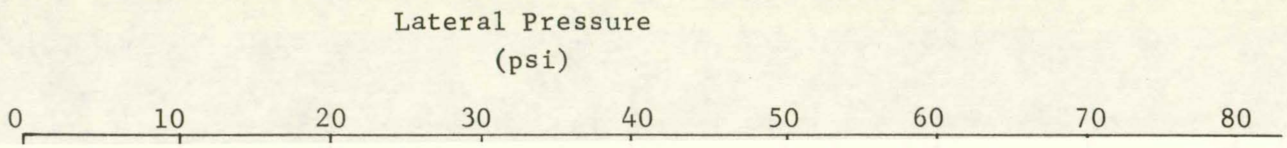
Figure 8. Stress path plot for Garner stone treated with SS-1 emulsion

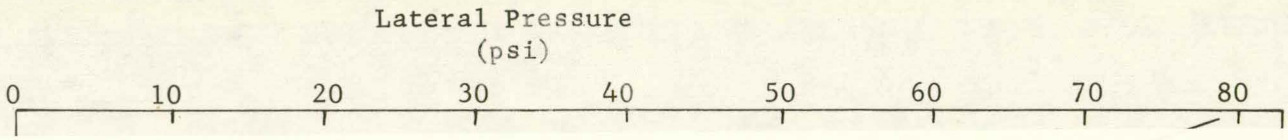
Lateral Pressure
(psi)

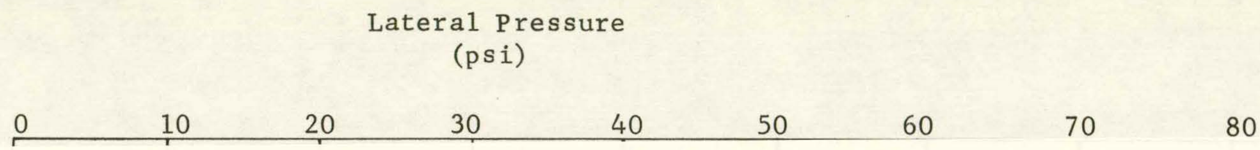
0 10 20 30 40 50 60 70 80

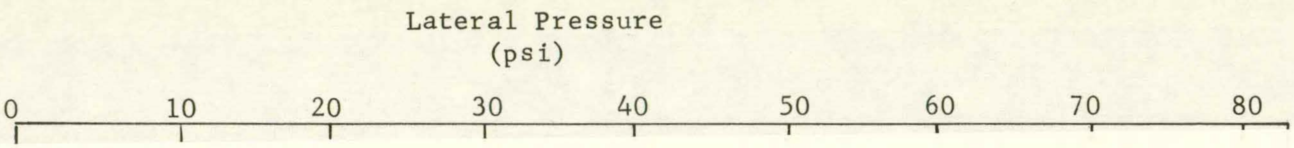
Lateral Pressure
(psi)


Lateral Pressure
(psi)
0
10
20
30
40
50
60
70
80


Lateral Pressure
(psi)
0
10
20
30
40
50
60
70
80


Lateral Pressure
(psi)
0
10
20
30
40
50
60
70
80


Lateral Pressure
(psi)
0
10
20
30
40
50
60
70
80


Lateral Pressure
(psi)
0
10
20
30
40
50
60
70
80

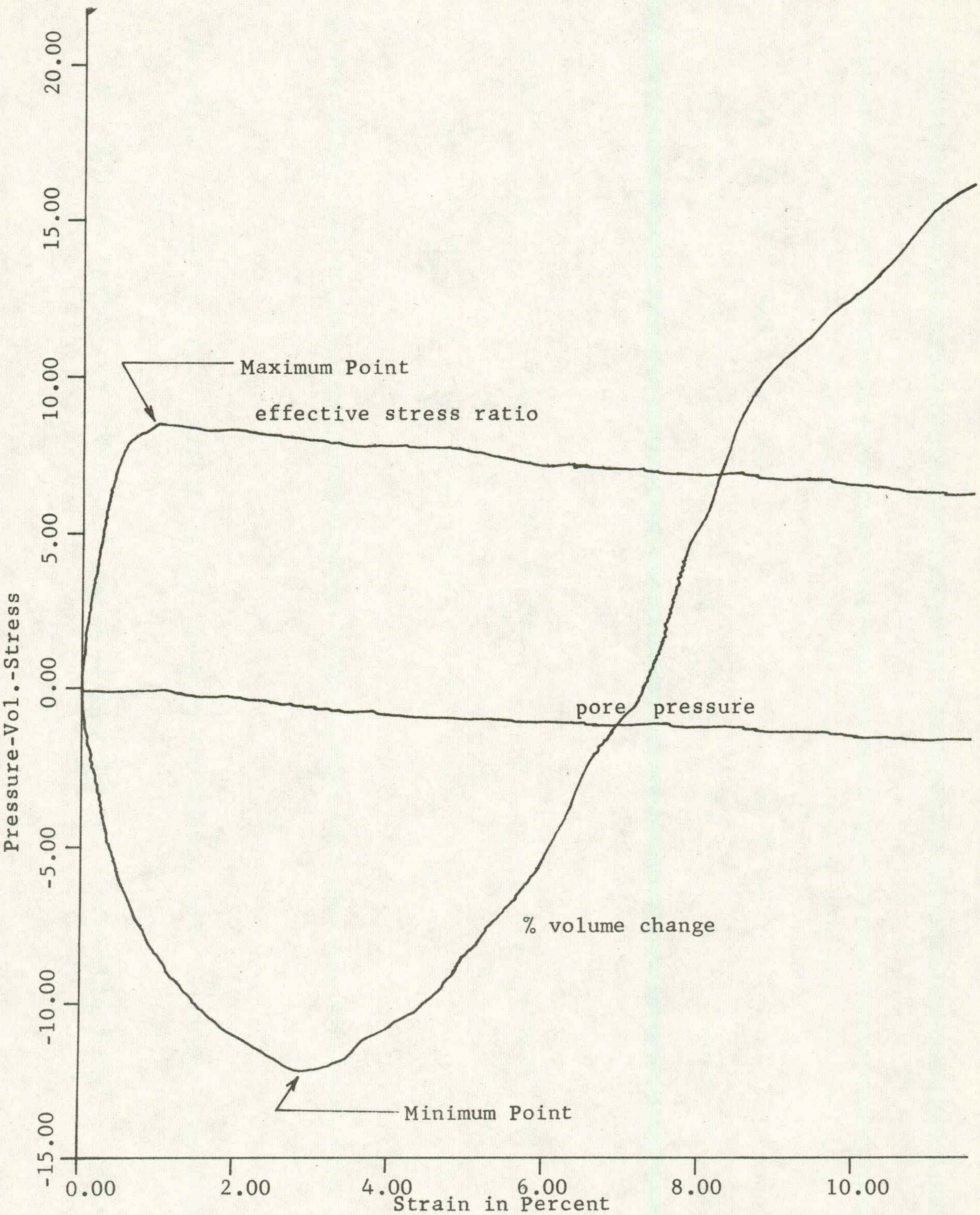


Figure 16. IBM 1627 plotter graph of effective stress ratio, pore water pressure, and percent volume change versus percent axial strain for a Bedford specimen treated with 4% asphalt cement and tested triaxially at 10 psi lateral pressure.

Table 5. Densities, Moisture contents, and shear strength parameters for bituminous treated and untreated Bedford stone

Additives Used	No. of Tests	Lateral Pressure	Average Dry Density	Average Moisture Content (%)	Mohr Envelope		Stress Path		Bureau of Reclamation	
					Φ'	c'	Φ'	c'	Φ'	c'
No additive	7	5,20,30, 40(2), 60 80	127.2±1.2*	10.1±0.7	46.2^{o}	7.0	45.5^{o}	6.6	45.7^{o}	6.7
SS-1 Emulsion (3.0% Asphalt)	5	10,20,40, 60,80	124.6±1.5	8.0±0.5	39.5^{o}	15.5	39.6^{o}	15.8	38.6^{o}	16.0
4,0% Asphalt Cement	6	10,20,30, 40,60,80	119.6±0.8	N-A**	41.2^{o}	10.5	41.6^{o}	10.7	41.0^{o}	11.8
4.0% Asphalt Cement (Sample less -200 Material)	6	10,20,30, 40,60,80	119.9±0.7	N-A**	38.9^{o}	17.0	39.0^{o}	15.4	38.9^{o}	17.0

*The (±) indicates maximum variation from the average for any test specimen

**Not applicable

Table 6. Densities, moisture contents, and shear strength parameters for bituminous treated and untreated Garner stone

Additives Used	No. of Tests	Lateral Pressure	Average Dry Density	Average Moisture Content (%)	Mohr Envelope		Stress Path		Bureau of Reclamation	
					Φ'	c'	Φ'	c'	Φ'	c'
No additive	10	10(2), 20(2), 30(2), 40(2), 60,80	145.4 ± 1.6*	6.8 ± 0.6	50.2°	11.0	49.8°	13.9	49.3°	14.2
SS-1 Emulsion (3.0% Asphalt)	8	5,15,20, 25.30,40, 60,80	142.9 ± 1.5	5.4 ± 0.89	45.3°	18.7	45.5°	16.9	45.5°	17.3
4.0% Asphalt Cement	6	10,20,30, 40,60,80	141.6 ± 1.1	N-A**	42.9°	16.8	42.9°	16.0	43.7°	15.9

* The ($\pm$) indicated maximum variation from the average for any test specimen

** Not applicable

Table 7. Densities, moisture contents, and shear strength parameters for bituminous treated and untreated Gilmore stone

Additives Used	No. of Tests	Lateral Pressure	Average Dry Density	Average Moisture Content (%)	Mohr Envelope		Stress Path		Bureau of Reclamation	
					Φ'	c'	Φ'	c'	Φ'	c'
No additive	6	10,20,30, 40,60,80	133.2 ± 1.8*	6.9 ± 0.7	46.6^{o}	12.0	46.2^{o}	13.4	46.2^{o}	13.2
SS-1 Emulsion (3.0% Asphalt)	6	10,20,30, 40,60,80	132.8 ± 1.1	5.0 ± 0.9	41.6^{o}	17.9	40.6^{o}	19.4	41.6^{o}	17.9
4.0% Asphalt Cement	6	10,20,30, 40,60,80	130.8 ± 1.2	N-A**	42.4^{o}	14.4	42.4^{o}	14.4	42.3^{o}	15.8

*The ($\pm$) indicates maximum variation from the average for any test specimen

**Not applicable

Table 8. Marshall stability test of Bedford sample treated with 4% asphalt cement

Sample No.	Density	Load lbs.	Flow Meter
B-1	122.9	169.3	8
B-2	119.3	164.3	8
B-3	121.7	152.7	9
B-4	117.9	142.4	14
B-5	122.1	153.3	7
B-6	119.2	175.9	9
B-7	117.7	150.7	14
Average	120.1	158.3	9

Table 9. Marshall stability test of Bedford sample (less -200 portion) treated with 4% asphalt cement

Sample No.	Density	Load lbs.	Flow Meter
B-1-a	122.3	208.3	8
B-2-a	122.2	178.5	8
B-3-a	119.8	159.9	9
B-4-a	120.3	205.3	11
B-5-a	121.9	171.3	10
B-6-a	122.5	204.1	8
B-7-a	121.5	162.4	6
Average	121.5	184.2	9

Table 10. Marshall stability test of Garner sample treated with 4% asphalt cement

Sample No.	Density	Load lbs.	Flow Meter
A-1	142.6	274.7	11
A-2	142.6	216.1	9
A-3	142.4	215.2	11
A-4	139.9	215.5	8
A-5	144.1	264.7	11
A-6	142.5	220.0	8
A-7	144.8	228.0	9
A-8	142.4	185.4	10
Average	142.6	227.4	8

Table 11. Marshall stability test of Gilmore sample treated with 4% asphalt cement

Sample No.	Density	Load lbs.	Flow Meter
G-1	133.7	230.8	10
G-2	133.1	176.6	11
G-3	132.4	252.4	7
G-4	130.1	164.3	10
G-5	130.0	141.3	12
G-6	132.4	325.6	11
Average	131.9	215.1	10