Engineering Research Institute

special report

Stability of Granular Base Course
Mixes Compacted to Modified Density

by

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Contribution No. 66-15
of the
Soil Research Laboratory
Engineering Research Institute

Iowa Highway Research Board Project HR-99
S. R. Roberts, Research Engineer

INTRODUCTION

Stability of a granular base course mix is an intangible quality that allows it to stand firmly in place, having durability or permanence, as well as strength. A granular base mix is said to be stable if it meets specific requirements to resist traffic loads without rupture or displacement throughout the expected life.

The stability of a granular base course mix is influenced by particle size distribution, particle shape, density, cohesion, and internal friction. Density is the factor most readily modified by mechanical means of compaction. The objective in compacting a soil is to improve its properties, and in particular, to increase its strength and bearing capacity, reduce its compressibility and decrease its ability to absorb water.

In 1933, R. R. Proctor showed that the dry density of a soil, obtained by a given compactive effort depended on the amount of water contained in the soil during compaction. For a given soil and a given compactive effort there is one optimum moisture content that will result in a maximum dry density. A standard compactive effort based on Proctor's work was adopted for use in construction control (19). Standard Proctor density (ASTM Designation D698-58T, AASHO Designation T99-57) can be considered as a laboratory density that is comparable to the density obtained in the field by compaction equipment producing equivalent energy.

The landing gear loads imposed by heavy military aircraft developed during the second world war, the onset of the jet age, and the rapid growth of truck traffic necessitated that the stability of granular base course mixes be improved. The most direct improvement of stability was to increase the density to greater than standard Proctor. Developments in the field of compaction equipment also made greater densities more attainable. Modified Proctor density (ASTM Designation D1557-58T, AASHO Designation T180-57) has thus been adopted for use on airfields, embankments, earth dams, etc. The compactive effort required for modified Proctor density is approximately 4.5 times that of standard.

The mixed service record of Iowa's granular base course materials stimulated research into the factors affecting the stability of these materials. The Engineering Research Institute at Iowa State University, under contract with the Iowa Highway Research Board, Iowa State Highway Commission, and the Bureau of Public Roads, U. S. Department of Commerce, has been conducting laboratory studies of the "Factors Influencing Stability of Granular Base Course Mixes."

The research reported here was undertaken to evaluate the various factors, including shearing strength, which influence the stability of crushed stone base course mixes compacted to modified Proctor density. Evaluation of the effect of gradation and mineralogy of fines on shearing strength of granular base course materials compacted to standard Proctor density is being conducted as another portion of the total project. Other related studies are concerned with improving the qualities of the materials with organic and inorganic chemical stabilization additives.

The predominant test procedure used throughout the project investigation has been the consolidated-undrained triaxial shear test with pore pressure and volume change measurements.

REVIEW OF LITERATURE

Stabilization by compaction has been practiced throughout the ages. Paths that had been trampled by animals were used by man who no doubt knew that trampling gave strength to the soil.

With the development of the wheel and the use of animals to pull wheeled vehicles, trampled animal paths changed from convenience to necessity as man found that there was much terrain over which he could not travel due to insufficient soil strength. During a period of rain he probably became aware of the fact that the soil strength was highly influenced by moisture. Since animal paths did not always lead man to where he wanted to go and because they were not always passable, man was forced to develop his own network of roads. The effectiveness of animals as compactors was not forgotten, however, and much use was made of them to compact the soils of newly found roads. Many ingenious ways to insure trafficability were developed. Brush, logs, stone, gravel and nearly any available material were used as means of improving the stability of roads.

The art of road building developed rather quickly and adequately kept pace with the development of transportation, until the advent of the automobile when tempo quickened and man promptly fell behind. Throughout this period the science of road building developed slowly. Modifying a well known parable, "The first shall come last and the last shall come first," man started at the top and worked to the bottom. The fundamentals of concrete and bituminous pavements are well documented. The fundamentals of base, sub-base, and subgrade are just emerging.

Soil Compaction

Road construction at the beginning of the twentieth century involved very little earth work and no thought was given to compaction except for some rolling of the surface layer. Development of earth moving equipment led to an increase in the depth of cuts and heights of fills. These fills were not compacted, but allowed to settle for a period of time. The rapid increase in automobiles in the 1920's produced a demand for more roads, which in turn demanded that fill "settlement" time be reduced. As a result, fills were placed in layers, sometimes moistened, and compacted, either by hauling equipment or rollers, to hasten settling.

Changing requirements of construction demanded that the engineer provide some measure of compaction. The first reported investigation of compaction was done by the California Division of Highways in 1929. Results of this investigation were never published except through departmental publications (19). The first widely published material on the fundamentals of soil compaction, by R. R. Proctor, in 1933, contained the results of an intensive study by the Bureau of Waterworks and Supply of the City of Los Angeles on the influence of compaction on shear strength and permeability (26, 27, 28, 29).

Proctor's work was much more than the relationships between moisture content, density, and compactive effort. The purpose of the work was to compact the soil to the density which by actual test was essential to obtain water-tightness and stability of a dam. The compaction standard set by Proctor (29) was one which gave a minimum penetration needle resistance (when saturated) of 300 psi. This value of 300 psi was selected because the soil would then have twice the penetration resistance required to permit loaded-truck travel when fully saturated. Proctor's reason for compacting was: "The compacted soil containing the least voids will require the less water to saturate it; therefore, when it is saturated it will have less lubrication between the soil particles and consequently less plasticity and greater stability" (26).

Following the publication of Proctor's report, numerous studies were made to increase the knowledge of the principles of compaction. Various factors influencing the maximum dry weight and optimum moisture content as determined in the compactive test have been thoroughly investigated. Included were: size and shape of the mold; number of layers; type, magnitude, and distribution of compactive effort; gradation, size, shape and type of aggregate; and degradation. These studies resulted in several different methods being adopted by different agencies. Standardization of the test was made by the American Association of State Highway Officials (AASHO) in 1938 and by the American Society for Testing and Materials (ASTM) in 1942 (20).

These early standards provided adequate maximum unit weights for highway construction. During World War II, the U. S. Army Corps of Engineers modified the test to produce the higher densities required for airfield construction (10). During the 1950's it became evident that higher densities were required in highway structures due to increased wheel loads and number of repetitions of same. A compaction requirement similar to the one developed by the Corps of Engineers was thus adopted by AASHO, 1957 and ASTM, 1958, and is generally referred to as "modified density" (20).

In a panel discussion (2) on "Compaction of Soils," two requirements for pavements were emphasized: first, a density must be attained to achieve stability under traffic loadings, etc.; and second, a strength factor must be sustained to resist shear stresses produced by traffic. It was also noted that densification under traffic strengthens the soil, but the resulting settlement may be detrimental.

In a study of the ultimate density of granular soils beneath thin bituminous surfacings, conducted by the Saskatchewan Department of Highways, field densities were obtained and compared with standard and modified Proctor densities. Observations were that the ultimate density of granular base course material to a depth of six inches beneath bituminous surfacing was 100 per cent of modified density. The soaked California Bearing Ratio (CBR) value of these soils at standard density was 61 while the soaked CBR in situ was 91 (35).

Vibratory Compaction

Compaction of granular materials by the standard laboratory methods is frequently not suitable. Disadvantages are: a) the indicated maximum density is not as great as can be achieved readily in the field, b) the moisture-density curve in some cases is not well defined, and c) the repeated ramming results in degradation of the particles. Many investigations have shown that these materials can be satisfactorily compacted using vibratory methods (7).

At present there is no standard AASHO or ASTM laboratory compaction method involving vibratory procedures. Most organizations using vibratory laboratory compaction use varying methods. Several of these methods have been studied by ASTM for consideration in the future development of a standard. Conclusions by Felt were that coarse grain granular soils were most effectively compacted using vibratory table methods and that densification was facilitated by water, especially when drainage from the bottom of the mold was permitted (7).

All investigations have indicated that the variables affecting vibratory compaction of granular materials are: frequency, period of time vibrated, amplitude, surcharge weight, moisture content, and soil type. Each of these variables have a marked effect on the densities attained. Also, the size of mold and the method of placement in the mold may have some influence on the attainable densities (7, 18, 25).

A study was made by the Engineering Research Institute at Iowa State University to determine a laboratory compaction procedure which would produce a specimen suitable for triaxial testing of compacted crushed stones. Major criteria were uniform, controlled density with a minimum of degradation and segregation of the compacted crushed stones. Compaction procedures analyzed were standard AASHO-ASTM, static, vibratory, and drop hammer. Vibratory compaction met the criteria noted above (18).

Stability

It has long been thought that stability (strength) increases as density increases. However, the use of density as a criterion for total stability can be misleading as the relationship is extremely complex. Many other factors, such as gradation, particle shape, plasticity, soundness, permeability, and moisture content, must also be evaluated.

In discussing the stability of granular bases and sub-bases, Yoder (35) suggested that gradation, especially the amount of fines, was the most important factor. No consideration of moisture content, as one of the factors, was evident. Maximum CBR values were obtained at lower percentages of minus No. 200 sieve fraction than were required for maximum density. Greater compactive effort increased the CBR value, regardless of the percentages of minus No. 200 material. The effect of plasticity of binder soil (passing No. 40 sieve) was observed to lower the CBR value as the plasticity index increased, and was most pronounced for high binder contents.

The effect of moisture content is most pronounced on soil binder having high plasticities. Laboratory and field testing of fine grained soils by the U. S. Army Corps of Engineers, Waterways Experiment Station, have shown conditions where an increase in density has resulted in an increase of strength as measured by CBR for a given water content up to a certain density. Additional increases in density resulted in decrease of strength (9).

This phenomenon has been labeled as "overcompaction." As explained by Mullis (22) this reduction in strength with an increase in density is the result of an increase in the degree of saturation. Other explanations which were offered by Mitchell, Holtz, Turnbull, and Johnson (2) indicated the same condition of too much moisture for the compactive effort being used. It is generally believed that this condition will only exist in fine grained cohesive soils.

Seed and Monismith (30) investigated the relationship between density and stability of subgrade soils using a silty-clay tested by triaxial compression, CBR, and Hveem Stabilometer. Their conclusions were:

"1. The relationship between density and stability of soil depends on the criterion used to define stability: The greater the permissible strain before a sample is considered unstable, the greater is the possibility that an increase in density will cause an increase in stability.

2. For samples of two soils, a silty clay and a sandy clay, compacted by kneading action, an increase in density at a given water content caused an increase or a decrease in stability (for strains less than 10 per cent) depending on the water content and range of densities involved; however, at a constant degree of saturation, an increase in stability.

6. For saturated subgrade conditions, the higher the density of the subgrade the greater will be its stability. However, for partially saturated subgrades, the desirable density for maximum stability will depend on the water content of the subgrade and too high a density may have a deleterious effect on stability."

Since no strength specification is required in the standard or modified Proctor compaction methods, Mullis (23) has advocated that criterion be accepted for stability that utilized the relationship of pore space and its water content to the strength of the material. He proposed:

"The ability of earth materials, including minerals, rocks, and rock sediments, to resist forces tending to destroy them, depends on chemical composition, atomic arrangement, pore space and the material's percentage of saturation. The percentage of pore space in an earth specimen at a constant percentage of saturation is inversely porportional to its strength. When the percentage of pore space in a given earth specimen is maintained at a constant value, its percentage of saturation is inversely proportional to its strength.

In an effort to prove his hypothesis much published data were assembled and tabulated. With few exceptions, the results support the fact that change in strength of a specimen is due to change in pore space and its percentage of saturation. In contrast to an earlier statement, it was found that the same relationship existed for granular materials, though an increase in density always resulted in an increase in strength (22, 23).

The U. S. Bureau of Reclamation initiated an extensive research program on the triaxial shear characteristics of granular soils (16, 17). One investigation was on the relationship between shear strength of pervious granular soils determined by consolidated-drained triaxial shear tests and a) density, b) per cent gravel fraction retained on No. 4 sieve, c) maximum particle size, and d) particle shape. Materials used were a river gravel and a crushed stone.

Significant conclusions of this study were: a) The shape of the particle affected the angle of internal friction. The crushed stone had a higher friction angle than the river gravel; b) The angle of friction was influenced by the initial density. Crushed stone with an initial void ratio of .369 had a computed cohesion of 2 psi and friction angle of 38.65 degrees while specimens with an initial void ratio of .296 had values of cohesion and friction angle of 3.0 psi and 40.40 degrees, respectively; c) The size of particles above 3/4 inches had little effect on the shear strength, but as the percentage of gravel was increased, the shear strength increased. Above a percentage of 50-60 (depending on the maximum particle size) this trend was reversed and there was no increase in shear strength, but possibly a slight decrease. Care must be used in interpreting these results as shear strength was compared with specimens at the same "relative" density and not at the same density (dry weight per unit volume). All computed shear strength envelopes for the crushed stone showed cohesion varying between 2.0 to 3.0 psi (17).

The concept of relative density is believed by many to be a more valuable index of granular soils than density as some percentage of a standard. It is believed that there is a maximum density for each granular soil and this is influenced by gradation alone (7). The major drawback of this approach is that there is no uniform procedure for determining the maximum density.

The U. S. Bureau of Reclamation also investigated the relationship of shear strength of clayey soils with variable gravel content. Density was not varied, but controlled to 95 per cent of standard Proctor. Unconsolidated-undrained triaxial shear tests were performed at a rate of strain equal to 0.02 inches per minute and pore pressures were measured. Results indicated that for amounts of gravel up to 35 per cent, the soil had shear properties similar to the clay matrix. At 50 per cent gravel content the effect of the gravel was apparent as the friction angle increased about 7 degrees and cohesion decreased about 3.0 psi. A further increase in gravel content to 65 per cent raised the friction angle an additional 2 degrees while cohesion remained about the same. Thus, the friction angle increased from 24.2 to 34.2 degrees and cohesion decreased from 8.7 to 5.0 psi as the gravel content varied from 0 to 65 per cent. It was concluded that there is an abrupt change in shear strength with increase in gravel content, which may vary with type of clay matrix and gravel (16).

From the two preceding Bureau of Reclamation studies a significant comparison can be made from the results on pervious gravelly soils with sandy matrix, and the same gravel with clay matrix. For specimens containing the same percentages of gravel, and compacted to about the same per cent of standard Proctor density, the angle of internal friction of the sand matrix was 8 to 12 degrees higher than the clay matrix (16).

An investigation by Morris (21) evaluated the effect of particle shape and surface texture on the strength and stability of granular materials. Particle size was held constant to about 1/8 inch diameter. Conclusions of this investigation were:

> "1. It has been proved that both the shape and texture of the particles affect the frictional properties of an aggregate to about an equal degree and that a change in either may produce more than 30 per cent variation in the yield strength.
>
> 2. The chemical composition of the particles has little to do with the strength of an aggregate..."

The effects of surface texture, density, and gradation on strength characteristics of granular base course materials were investigated through triaxial shear tests for flexible pavement design in Indiana (36). Crushed limestone and pitrun gravel, compacted to 97 and 100 per cent standard Proctor density, were used. Gradations were prepared at the upper limit, medial, and lower limit of coarseness of AASHO specification B-1, Designation M56-42, for stabilized base course materials. Specimens were tested at 7, 14, and 20 psi lateral pressure, and the friction angle was determined for each of the above variables. Type of triaxial shear test and magnitude of cohesion, if any, were not reported. The following conclusion was made: "...that the angles of internal friction of the base course materials were not influenced to a marked degree by either compactive effort, gradation, or surface texture." Modulus of deformation, as determined by the straight-line portion of the stress-strain curves was increased with an increase in density, also with an increase in coarseness of gradation.

The National Crushed Stone Association (24) investigated dense graded aggregates for base courses by means of the triaxial shear test developed at the Texas Highway Department. Specimens were molded to modified Proctor density by conventional procedure and then vibrated with a surcharge of 3 psi for further densification.

Of the variables studied, the optimum per cent of fines passing No. 200 sieve was determined to be 9 per cent for a maximum particle size of 3/4 inches. The angle of internal friction decreased in order of 9, 4.5, 13, 1, and 20 per cent passing the No. 200 sieve. The modulus of deformation (defined as the tangent to the stress-strain curve of specimens tested at 20 psi lateral pressure) was 25,000, 26,000, 23,000, and 19,000 psi for 1 and 4.5, 9, 13, and 20 per cent passing the No. 200 sieve, respectively.

Computed values of cohesion existed for all shear strength envelopes. The author's explanation was:

> "The rupture envelopes cross the ordinate of zero normal stress at different points which is caused by the 'apparent cohesion' that exists in well graded mixes when moist. Aggregate interlock is greater for the gradings with the larger aggregate; the larger aggregate forms what may be described as 'obstacles' in the planes of failure which increases the strength of the mix."

Also concluded was that crushed stone was better than crushed gravel or rounded gravel. In addition, the investigation indicated that both shear strength and modulus of deformation decreased as plasticity of the fines increased.

Unconsolidated-undrained triaxial shear tests with pore pressure measurements were made by the Melbourne and Metropolitan Board of Works to supply shear strength data suitable for earth dam design (12). The effect of decreasing the initial degree of saturation was to increase cohesion with minimum variations in the friction angle. The effect of increasing density from standard Proctor to modified Proctor was similar to that observed with decreasing saturation.

"There are minor changes in the friction parameter, but they are small enough to have been caused by normal experimental errors. There are, however, very marked increases in cohesion parameters." When pore pressure was plotted against total stress, it increased as the total stress increased. The slope of this line was affected by both the degree of saturation and density. As saturation increased the slope became steeper and as density was increased the slope was lowered. Also noted was a critical stress point at which there was an abrupt increase in the slope. This point occurred at lower values of total stress as the degree of saturation was increased and higher value as the density was increased. This critical stress was low for fine grained soils, but for coarse textured soils it was not observed in the range of total stresses tested.

Triaxial shear studies by the Engineering and Water Supply Department, South Australia, also found that the friction angle remained the same for standard Proctor and modified Proctor densities and the cohesion increased as density increased (8).

Laboratory tests of the gravel and crushed stone used as base courses in the AASHO Road Test have been conducted (13). Molded specimens with variable amount of fines (passing the No. 200 sieve) and percentage of saturation were subjected to repeated loading and the corresponding deflections and rebounds were measured. The relationship of total deflection versus number of repetitions was greatly affected by saturation and to a lesser extent by the per cent of fines. The number of repetitions to produce a given deflection increased as saturation and per cent of fines decreased. The number of repetitions to produce 0.1 inch total deflection are summarized below.

Minus No. 200 Sieve, %	Degree of Saturation, %	Number of Repetitions
6.2	80	900
6.2	67	90,000
9.1	81	2,000
9.1	68	20,000
11.5	81	1,000
11.5	68	10,000

It is evident that the optimum percentage of fines passing through the No. 200 sieve is dependent on the degree of saturation.

MATERIALS

Three crushed stones were used in this project. Each was 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.

1. A weathered, moderately hard limestone of the Pennsylvania System, which outcrops about half of the state of Iowa. This was obtained from near Bedford, Taylor County, Iowa. Hereafter it will be referred to as the Bedford sample. Generally, the bedrock of the Pennsylvania System is of poor quality but it is the only economically available crushed stone in southwest, south, southeast, and central Iowa.
2. A hard, concrete quality, limestone of the Mississippian System which outcrops in a rather discontinuous and patchy band from north central to southeast Iowa. This was obtained from near Gilmore City, Humboldt County, Iowa. Hereafter, it will be called the Gilmore sample.
3. A hard dolomite of the Devonian System which outcrops in a wedge-shaped belt roughly parallel to the Mississippian System on the northern side. The bedrock of the Devonian System is consistent quality and is a valuable source of aggregate in Iowa. The sample was obtained from near Garner, Hancock County, Iowa. It will be referred to as the Garner sample.

All three stones were tested as obtained from the quarries without modification of their physical properties. In addition, the Bedford stone was tested after removal of the fines; i.e., particle sizes passing the No. 200 sieve, by dry sieving. This sample will be referred to as the Bedford + 200. Of the three stones, the Bedford has the poorest base course service record.

Chemical and mineralogical tests of the three stones are presented in Tables 1, 2, and 3. Representative samples of each material 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 was used for quantitative measurement of pH, cation exchange capacity (C.E.C.) and hydrochloric acid soluble and non-soluble minerals. Table 4 presents the engineering properties of the three materials (18).

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

Stone Des.	Calcite	Dolomite	Quartz	Feldspars	Calcite/Dolomite Ratio*
Bedford	Pred.	Small Amt.	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 Amt.
Garner	None	Small Amt.	Pred.	Second Pred.	Large Amt.
Gilmore	None	None	None	Pred.	Small Amt.

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 Material, %	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 (0.001 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	10.8	7.6	9.3
Dry density, pcf.	128.0	140.5	130.8
Modified AASHO-ASTM Density:			
Optimum moisture content, % dry soil weight	8.0	5.4	5.7
Dry density, pcf.	133.5	147.6	140.8
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

METHODS OF INVESTIGATION

Stability of granular base course mixes encompasses a wide range of behavior, as can be seen from the general definition on page 1. Because of the difficulty of measuring stability in the field a number of laboratory tests have been developed and correlated with field performances.

Probably the most widely used index of stability is the California Bearing Ratio. This is based on the resistance of the material to penetration by a plunger. Another widely used index to stability is the Resistance Value as measured by a Hveem Stabilometer. This is a closed system triaxial compression test in which a verticle load is applied at a constant strain and lateral pressure transmitted by the specimen is recorded.

A third test which is used as an index to stability is triaxial shear. A specimen is subjected to a confining lateral pressure and is then sheared by axial load. This test was devised as a research tool and can produce stress and strain conditions that closely duplicate field conditions. For fundamental research on soils and testing programs in which the time required for individual tests is not the controlling factor, this type of apparatus is generally recognized as the best presently available.

The consolidated-undrained triaxial shear test with total volume change and pore water pressure measurements was chosen to evaluate the stability of granular base course mixes compacted to modified Proctor density.

Modified Proctor Density

Modified Proctor density was determined in accordance with ASTM Designation D1557-58T, AASHO Designation T180-57, Method C (1, 3). The test is performed in a cylinder having a volume of 1/30 cubic foot using 25 blows of a 10 pound hammer dropped from a height of 18 inches on each of five equal layers of material. The cylinder is 4.0 inches in diameter and 4.59 inches high. The face diameter of the hammer is 2.0 inches. The total compactive effort is 56,250 foot pounds per cubic foot. In comparison, the compactive effort of the standard Proctor test is 12,317 foot pounds per cubic foot.

To determine the moisture-density relationship of the three stones, representative samples, each large enough to produce one modified density specimen, were air dried. Increasing amounts of distilled water were added to produce specimens at different moisture contents. The samples were thoroughly mixed by hand. They were covered by a damp cloth and allowed to stand for 30 minutes to permit more complete absorption of the moisture. Specimens were then compacted in accordance with the procedure described above.

Following compaction, each specimen was weighed, extruded by a hydraulic jack, and visually examined for segregation. Moisture content samples were taken from the top, middle, and bottom of the specimens. The remainder was retained for mechanical analysis in order to determine the amount of degradation of the compacted specimen. The moisture content samples were placed in an oven and heated to $110^{\circ}C$ for a minimum of 18 hours.

Moisture density results are given in Table 4 and the moisture density curves for each of the three stones are presented in Figure 1. The moisture density relationship was not determined for the Bedford + 200 as it was desired that the Bedford + 200 specimens be prepared at the same density and moisture content as the Bedford.

Triaxial Specimen Preparation

Previous studies have indicated that granular materials are more suitably compacted using vibratory methods and that the obtainable density is dependent on the combination of surcharge weight, amplitude, frequency, and period. This method was chosen for the compaction of the triaxial specimens (4 inch by 8 inch cylinders) to the modified Proctor density previously determined by AASHO/ASTM procedures. A Syntron, Model V-60, electromagnetic vibrator table at a constant frequency of 3600 vibrations per minute was used. The amplitude could be varied with a rheostat graduated from 0 to 100.

Hoover (18) found that this size triaxial specimen could be compacted to standard Proctor density with little or no particle degradation and segregation by the following combination of factors:

1. Rheostat dial setting of 90, for an amplitude of 0.368 inch.
2. Period of vibration of two minutes.
3. Surcharge weight of 35 pounds, or 2.78 psi.

Also noted was that degradation decreased as the surcharge weight was increased.

To determine the combination of factors necessary for vibratory compaction to modified Proctor density without degradation and segregation

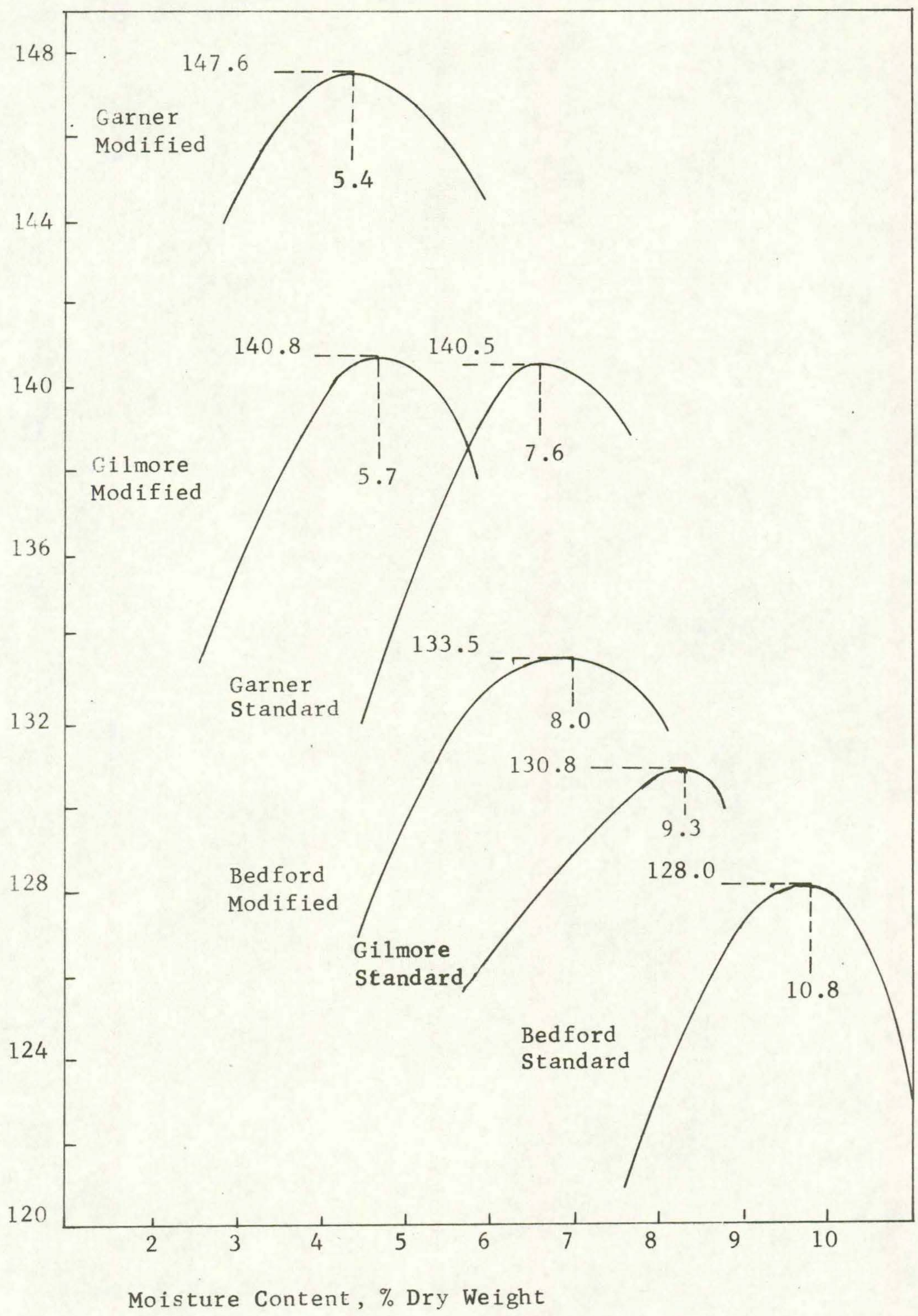

Figure 1. Moisture-density relationship of crushed stones

of particle sizes, the relationship between density and surcharge weight was investigated. The rheostat dial setting and the period of vibration were held constant at 90 and 2 minutes, respectively.* A surcharge weight of 105 pounds (8.35 psi) was determined to produce modified Proctor density. Figure 2 shows vibratory table, mold, and surcharge weight used for the preparation of the triaxial specimens.

No moisture-density relationship was determined for the vibratory compaction. However, initial moisture content greater than the optimum obtained by modified AASHO/ASTM procedures was required. The moisture content after vibration was 1.1 per cent greater than the AASHO optimum for the Bedford, 0.3 per cent for the Garner, and at the optimum for Gilmore. This condition is in agreement with the results of vibratory compaction reported by Felt (7).

Preparation of the triaxial shear specimen began by air drying sufficient crushed stone for a 4" x 8" specimen plus a 400 gram moisture content sample. Distilled water was added to produce the initial moisture content as determined above. All mixing was accomplished by hand to prevent degradation of the material, after which the mix was allowed to stand for 30 minutes. The 400 gram moisture content sample was then removed and the remaining mix was added to the mold in five equal layers, each layer being rodded until refusal with a 5/8 inch diameter tapered point steel rod. The surcharge weight of 105 pounds was placed on the specimen and compaction was accomplished in accordance

*A rheostat dial setting of 90 is considered to be the maximum limit of the vibratory table used. The optimum period of vibration is considered to be 2 minutes. At this time a thick slurry of fines and water starts to ooze from the top and bottom of the specimen.

with the previously discussed combination of factors.

After removal from the vibrator table, each specimen was extruded from the mold by hydraulic jacking. Following weighing and measuring of height, each specimen was wrapped in a double layer of Saran Wrap and aluminum foil and placed in a curing room, at near 75^{o}F and 100 per cent relative humidity for a minimum of 48 hours and a maximum of seven days. Purpose of this curing was to allow the moisture to equilibrate throughout the specimen.

Mechanical Analysis

Particle size distribution of each of the three crushed stones was determined by mechanical and hydrometer analysis in accordance with ASTM Designation D 422-63 (3). These tests were conducted on each material in the following conditions:

1. Uncompacted.
2. Compacted to modified Proctor density by AASHO/ASTM method.
3. Compacted to modified Proctor density by vibratory method.

The gradations determined are given in Table 5.

Triaxial Machine

The triaxial machine used in this investigation was developed by the Engineering Research Institute, Soil Research Laboratory and built by the Engineering Research Institute Shop. The unit consists of two bays capable of testing two specimens simultaneously under different lateral pressures and drainage conditions.

Table 5. Mechanical analysis of compacted crushed stone specimens.

Stone, type	Per Cent of Total Gravel[a]	Sand[b]	Silt[c]	Clay[d]	Colloids[e]
Bedford					
Uncompacted	73.2	12.9	8.4	5.5	1.7
Modified AASHO/ASTM	65.8	15.0	11.8	7.4	1.9
Modified Vibratory	68.8	14.6	10.2	6.4	2.0
Garner					
Uncompacted	61.6	26.0	10.2	2.2	1.4
Modified AASHO/ASTM	55.7	29.8	11.4	3.1	1.8
Modified Vibratory	58.3	28.0	11.0	3.0	1.5
Gilmore					
Uncompacted	66.8	23.3	5.9	4.0	0.9
Modified AASHO/ASTM	62.0	26.2	7.2	4.6	1.6
Modified Vibratory	64.3	24.8	6.9	4.0	1.4

[a]Particle size greater than 2.00 mm.

[b]Particle size 2.00 to 0.074 mm.

[c]Particle size 0.074 to 0.005 mm.

[d]Particle size less than 0.005 mm.

[e]Particle size less than 0.001 mm.

Rate of strain is variable between 0.0001 and near 0.1 inch per minute. The set rate is constant within 1/2 of 1 per cent under all loads, as produced through combination of a Dynamatic Adjusto Speed Motor controlled by a Dynamatic Silicon Controlled Rectifier, Turner Two-speed Transmission, and Link Belt Worm Gear Speed Reducer. A maximum axial load of 11,000 pounds can be transferred to the specimen through a calibrated proving ring. The vertical deflection of the specimen is measured with a dial gage extensometer.

Lateral pressures can be applied to a specimen within a plexiglass cell by an air over liquid system or air pressure only. This pressure can be varied between 0 and 100 psi and is held constant throughout a test by means of a diaphragm regulator within 0.3 psi. Pore water pressure developed during testing is measured at the base of specimen by a Karol-Warner Model 53-PP pore pressure device which operates on the null-balance principle, measuring both positive or negative pore water pressures.

Volume changes can be obtained when water is used in the cell. This is determined by maintaining a constant water level within the cell and measuring the amount of water that flows from or into a graduated tube that is under equal pressure. The triaxial unit is illustrated in Figure 3.

Consolidated-Undrained Triaxial Shear Test

A specimen obtained from the curing room was weighed and measured, and placed in the triaxial cell. Each specimen was sealed in a rubber membrane of uniform 0.025 inch thickness with saturated 1/2 inch thick

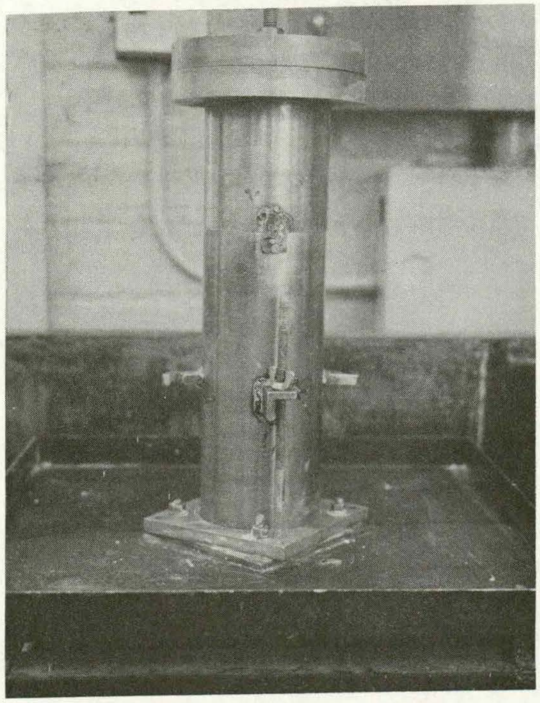

Figure 2. Vibratory compaction apparatus

Figure 3. Triaxial shear machine

corrundum porous stones on top and bottom. The cell was filled with water to a fixed height, consolidating pressure was applied, and drainage permitted through the base of the specimen during the consolidation-phase. Volume change measurements were made, together with vertical deflection, at time intervals of 1, 2, 4, 9, 16, 25, and 36 minutes. Primary consolidation of the specimens tested was determined from a plot of volume changes versus the square root of time using the method developed by Taylor (31). A time of 36 minutes was found adequate for primary consolidation of all specimens.

After consolidation the specimen was sheared with the drainage valve closed and pore pressure device opened. Axial load was applied at a constant vertical deformation rate of 0.01 inch per minute. Volume change, pore pressure, and axial load from proving ring deflection were recorded every 0.025 inch of vertical deflection. All tests were carried to a vertical deflection of one inch which was well beyond the point of shear failure. Following shear, each specimen was removed from the cell, weighed, and divided into top, middle, and bottom segments for moisture content determination using the procedure discussed previously.

Calculation and reduction of the raw shear test data was accomplished by using an IBM 7074 computer program and a 1627 L-Comp plotter through the Iowa State University Computer Center. Provided were a printout of all relations and a plot of effective stress ratio, volume change, and pore pressure versus per cent strain. This data was then used for later analyses.

Specimens of each stone were consolidated and sheared at lateral pressures of 10, 20, 30, 40, 60, and 80 psi. Duplicate specimens were tested for the Bedford stone.

ANALYSIS OF RESULTS

Normal methods of interpretation of triaxial tests use a criterion of failure defined by consideration of stresses only. When a triaxial test is carried out on granular soils for use in highway base course design, such methods of investigation may result in obtaining only limited analyses of the total stability of these methods. A concept of failure is proposed herein which utilized the stress-volume change-strain relationship, to more adequately define the failure criterion of granular base course mixes. Justification for this failure criterion will be shown in the succeeding sections.

Shear Strength Criterion

It is common practice to express the shear stress/strength of soil with the Coulomb equation:

$$\tau = C + \sigma \text{ Tan } \phi \tag{1}$$

in which τ is the shear stress, C is the cohesion parameter, σ is the normal stress on the failure plane, and ϕ is the angle of internal friction. This is an empirical equation and no physical significance is necessarily attached to the parameters.

When drainage is prevented in a triaxial shear test, not all of the load imposed on the specimen is carried by grain to grain contact, i.e., intergranular pressure. Some portion is carried by the void fluid, air and water, and is called hydrostatic excess pressure or simply pore pressure. Terzaghi (32) was the first to recognize this and proposed this correction:

$$\overline{\sigma} = \sigma - \mu \qquad (2)$$

in which $\overline{\sigma}$ is the effective stress on the particles, σ is the total stress and μ is the pore pressure. Since friction is caused by intergranular pressure, equation 1 is modified for effective stress analysis.

$$\tau = C' + \overline{\sigma} \text{ Tan } \phi' \qquad (3)$$

In considering the effective stress on all planes through a point, the various planes will, in general, be found to have both normal stress and shearing stress acting on them. All details that pertain to stress at a point may be represented graphically by a circle known as the Mohr's circle in which normal stress is represented by the abscissa and shearing stress by the ordinate. Any point on the circle represents the coordinates of stress on some plane.

In triaxial testing with the effective lateral pressure $\overline{\sigma}_3$ constant, and the effective vertical pressure $\overline{\sigma}_1$ increasing, a series of Mohr's circles can be drawn as the specimen is stressed to failure. By constructing the Mohr's circles of several specimens tested at different lateral pressures a line drawn tangent to the failure circles is defined by the shear stress equation, where the slope is tangent ϕ' and the ordinate intercept is C'. This line is called the Mohr-Coulomb shear strength envelope. A graphical definition of the Mohr's circle and shear strength envelope is given in Figure 4a.

If the results of a large number of triaxial tests are plotted on a single Mohr-Coulomb diagram, the failure envelope may be obscured by the presence of many circles. Therefore, the data may be plotted in a modified form, referred to herein as the "stress path," in which 1/2 $(\bar{\sigma}_1 - \bar{\sigma}_3)$ is plotted against 1/2 $(\bar{\sigma}_1 + \bar{\sigma}_3)$ (5). Two advantages of this plot are: first, as a specimen is stressed to failure (increasing radius of the Mohr's circle) each circle can be represented by a single point and the stress path can be visualized; and second, the failure points for a number of specimens are exactly identified. The angle of slope of the resulting failure envelope is designated as A and the ordinate intercept as X. These "stress path" parameters can be converted to the standard parameters (ϕ' and C') using the equations:

$$\text{Sin } \phi' = \text{Tan } A \tag{3}$$

$$C' = X/\text{Cos } \phi' \tag{4}$$

A graphical definition of the "stress path" (as used in this report) as a specimen is stressed to failure is given in Figure 4b.

Because of the complex nature of a soil's shear strength, the envelope for either the Mohr-Coulomb or "stress path" analysis may not fit the data. A "least squares" fit may then be used to determine the equation of the envelope.

The precise point of failure of a triaxial specimen of soil has been defined in a number of different ways. Two of the most common criteria of failure are maximum deviator stress, $\bar{\sigma}_1 - \bar{\sigma}_3$, and maximum effective stress ratio, $\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\bar{\sigma}_3}$. Holtz advocated that when pore pressure existed in specimens, the maximum effective stress ratio should be the true failure criterion (14).

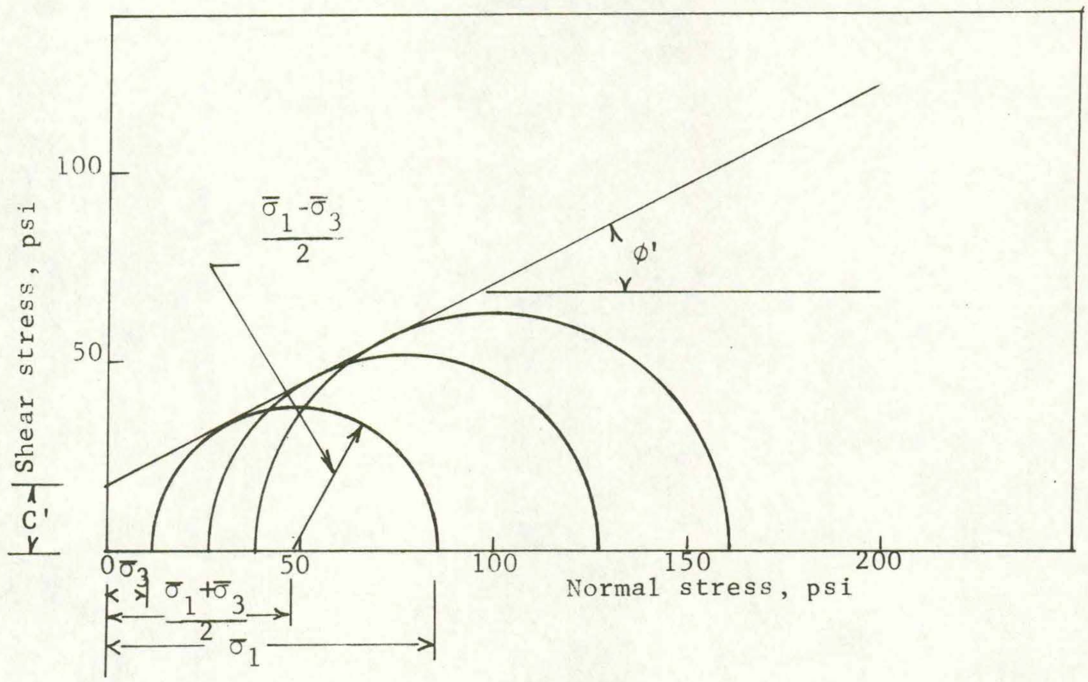

Figure 4 (a). Mohr envelope analysis

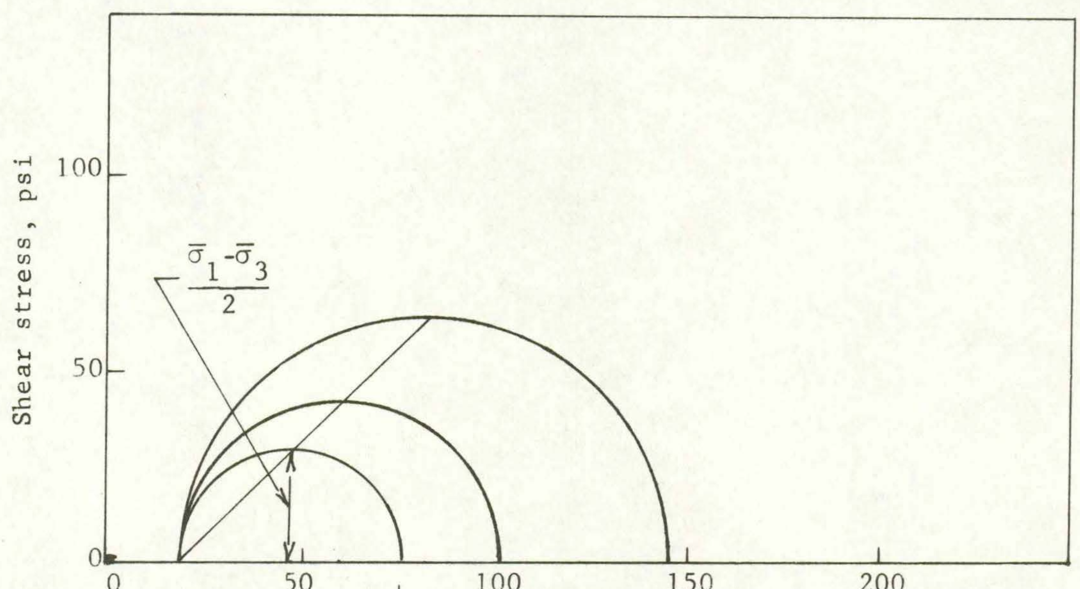

Figure 4 (b). Stress path representation by Mohr circle maximum ordinates

One must look at the mechanics of shear in a triaxial specimen of crushed stone for a better understanding of failure. As the axial load is applied a vertical deflection takes place. Since Poisson's ratio for soil is generally believed to be greater than zero, the specimen tends to deform laterally. This is resisted by the applied lateral pressure and the specimen compresses. This compression continues as the axial load is applied until some minimum volume is reached, the latter being a function of the applied lateral pressure and placement density. The pore pressure increases during this loading, causing a reduction in the effective vertical and lateral pressures. When this minimum volume is reached, the effective lateral pressure is not enough to resist lateral deformation. For lateral deformation to occur the particles must slide over one another. In granular soils this produces a significant volume increase as the particles rise up and over each other. This volume increase causes the pore pressure to reduce.

In his analysis, Holtz (14) gives the following:

> "It seems logical that, as the effective lateral pressure increases, because of decreased pore pressure, some gain in axial strength can be anticipated, even though failure might have already begun. That is, the axial strength should increase because the effective lateral support is increased. The maximum principal stress ratio appears to represent the most critical stress condition or the point of incipient failure under variable effective axial and lateral stresses. A study of the volume change conditions during the tests indicates that specimens consolidate to some minimum volume, after which the volume increases as loading is continued. It is believed that the minimum volume condition, or some point near this condition, indicates the condition of incipient failure. That is, the condition at which consolidation ceases and the mass begins to rupture. The maximum pore-pressure condition should occur when the specimen has been consolidated to a minimum volume, because at this point the pore fluid has been compressed to the greatest degree.

In correlating the test data, it was found that the point of maximum pore pressure, and minimum effective lateral pressure, coincided very closely with the minimum volume and maximum principal stress-ratio points."

The authors agree wholeheartedly with the above point that the minimum volume condition, or some point near this condition, indicates the conditions of incipient failure for granular base courses. In correlating the test data of this research, it was found that the point of maximum pore pressure coincided very closely with the minimum volume and both of these occurred at a lesser per cent axial strain than the maximum effective stress ratio. The shear stress continues to increase as the volume increases until the maximum effective stress ratio is reached. From the stress path analysis it can be seen that the limiting Mohr's circle occurs at the maximum effective stress ratio.

A Mohr's circle is defined by the maximum deviator stress and is tangent to the failure envelope defined by the maximum effective stress ratio or at a slightly lower angle. This condition is shown in Figure 5 for the upper portion of one of the stress path diagrams. This increase in shear stress as the specimens expand from minimum volume to the point of maximum effective stress ratio is the result of the work done by the specimen in increasing its volume during shear. Taylor (31) discusses the interlocking effect in sands as:

"Interlocking contributes a large portion of the strength in dense sands; this phenomenon does not occur in very loose sands. The gradual loss of strength after the peak point is passed,..., may be attributed to a gradual decrease in interlocking which takes place because the sample is decreasing in density. The angle of internal friction, in spite of its name, does not depend solely on internal friction, since a portion of the shearing stress on a plane of failure is utilized in overcoming interlocking.

Interlocking can best be explained by consideration of strain

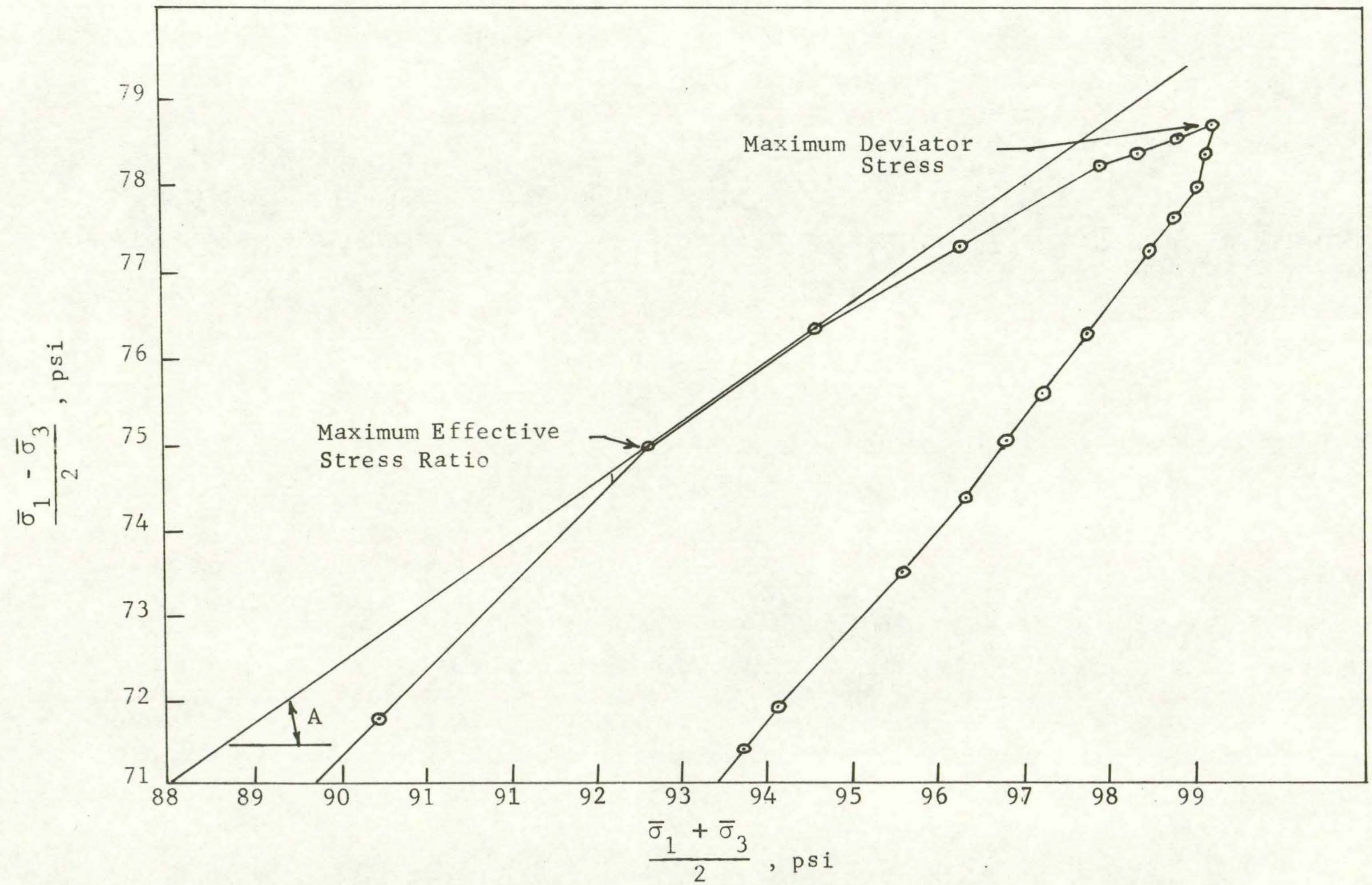

Figure 5. "Stress path" at points of maximum effective stress ratio and maximum deviator stress, Bedford specimen, 20 psi lateral pressure

> energy. Sands generally are undergoing increase in volume when the ϕ - obliquity condition is reached, and the part of the shearing stress that is acting to overcome interlocking may also be said to be supplying the energy that is being expended in volume increase.
> For the occurrence of the expansion, which is resisted by the applied lateral pressure, energy must be supplied in some way.

For fine grain soils, as used in Holtz's research (14), the effect of particle interlock is negligible and there is very little volume increase during shear with the point of minimum volume coinciding with the point of maximum effective stress ratio. In granular soils, such as crushed stones, the effect of particle interlock is magnified by a significant volume increase during shear as particle interlock is overcome. The maximum effective stress ratio occurs at some point during this volume increase with a shear stress greater than at minimum volume.

In granular base course design it is desired that the materials be compacted to such a density that the loads imposed by traffic cause little or no consolidation. This consolidation adds to the strength of the base but failure can occur due to excessive consolidation which causes pavement cracking and/or an undesirable riding surface. This is not a structural failure and will not be considered further.

Another type of failure is by shear deformation which involves movement out from under the traffic lane producing rutting and upheaval. It is felt that shear deformation is caused by the loss of lateral support due to the lateral deformation that starts to occur after minimum volume is reached.

An analogy can be made to a retaining wall to explain this loss of lateral support. If the wall is moved towards the soil a passive condition is developed and the force of the soil on the wall is large. If a force is added that causes the wall to move away an active condition is developed and the force of the soil on the wall is reduced.

One may view compaction as causing the passive condition to be developed by forcing the soil into a more dense state. An axial load causes further densification until the required lateral support is greater than the passive force that can be developed. Lateral deformation then occurs causing the lateral force to decrease to the active condition and eventually failure occurs.

In this report, initial analyses were made using the maximum effective stress ratio as the failure criterion. However, final analysis was made using minimum volume as the failure criterion as it is felt that this point is the critical condition at which failure occurs in crushed stone base course materials.

Granular soils are usually considered to be cohesionless, i.e., cohesion equals zero. In triaxial testing of sands this is generally observed to be true, but in triaxial testing of crushed stone and gravels the cohesion parameter is greater than zero. Most investigators explain this "cohesion" value as the effect of particle interlock.

> "The effect of particle interlock is considered to be of considerable importance. Particle interlocking is achieved through increased density, increase of gravel content and maximum particle size which results in improved gradation and increased sharpness of particle shape. We have found that the shear strength could not be analyzed by one shear test with a straight line envelope of limiting shear resistance passing through the origin of the normal stress-shear stress plot, because of the interlocking effect is registered as an intercept on the shear stress ordinate (called "c" although the material is cohesionless)..." (16).

Table 6. Shear strength parameters for the crushed stones

	Failure Criterion			
Material and	Maximum Effective Stress Ratio		Minimum Volume	
Compaction	ϕ', degrees	C', psi	ϕ', degrees	C', psi
Bedford-modified	43.6	13.46	44.8	1.28
Bedford + 200-modified	44.7	10.77	43.3	0.92
Bedford-standard	45.7	6.68	46.2	4.21
Garner-modified	48.6	18.30	48.6	3.73
Garner-standard	49.2	14.20	49.5	5.58
Gilmore-modified	46.8	18.07	46.6	8.75
Gilmore-standard	45.1	17.09	45.5	8.89

Shear Strength

A summary of all triaxial test data is presented in Appendix A, Tables 7 through 13. The shear strength parameters for the stones tested in this study are presented in Table 6. Included in all data presented are the results of the investigation on the same stones compacted to standard Proctor density (18) which hereafter will be referred to as Bedford standard, Gilmore standard, and Garner standard. The shear strength parameters were computed by using a "least squares" fit method developed by the Bureau of Reclamation (33).

The Mohr-Coulomb envelopes for all stones evaluated at the minimum volume condition are shown in Figure 6. "Stress paths" for the stones at modified density are plotted in Figures 7, 8, 9, and 10, including envelopes for both minimum volume and maximum effective stress ratio.

These results show that the angle of internal friction varied slightly from the point of minimum volume to the point of maximum effective stress ratio. Cohesion was increased significantly, reflecting the effect of particle interlocking. The change of compaction effort from standard to modified Proctor increased the amount of interlocking and a higher cohesion value is shown at maximum effective stress ratio.

The angle of internal friction at minimum volume was reduced by an increase in density from standard to modified Proctor for the Bedford and Garner stones and increased for the Gilmore. The change was 1.4 to 0.9 degrees reduction for the Bedford and Garner stones, respectively, with an increase of 1.1 degrees for the Gilmore. In general, this variance may be small enough to be accounted for by normal experimental

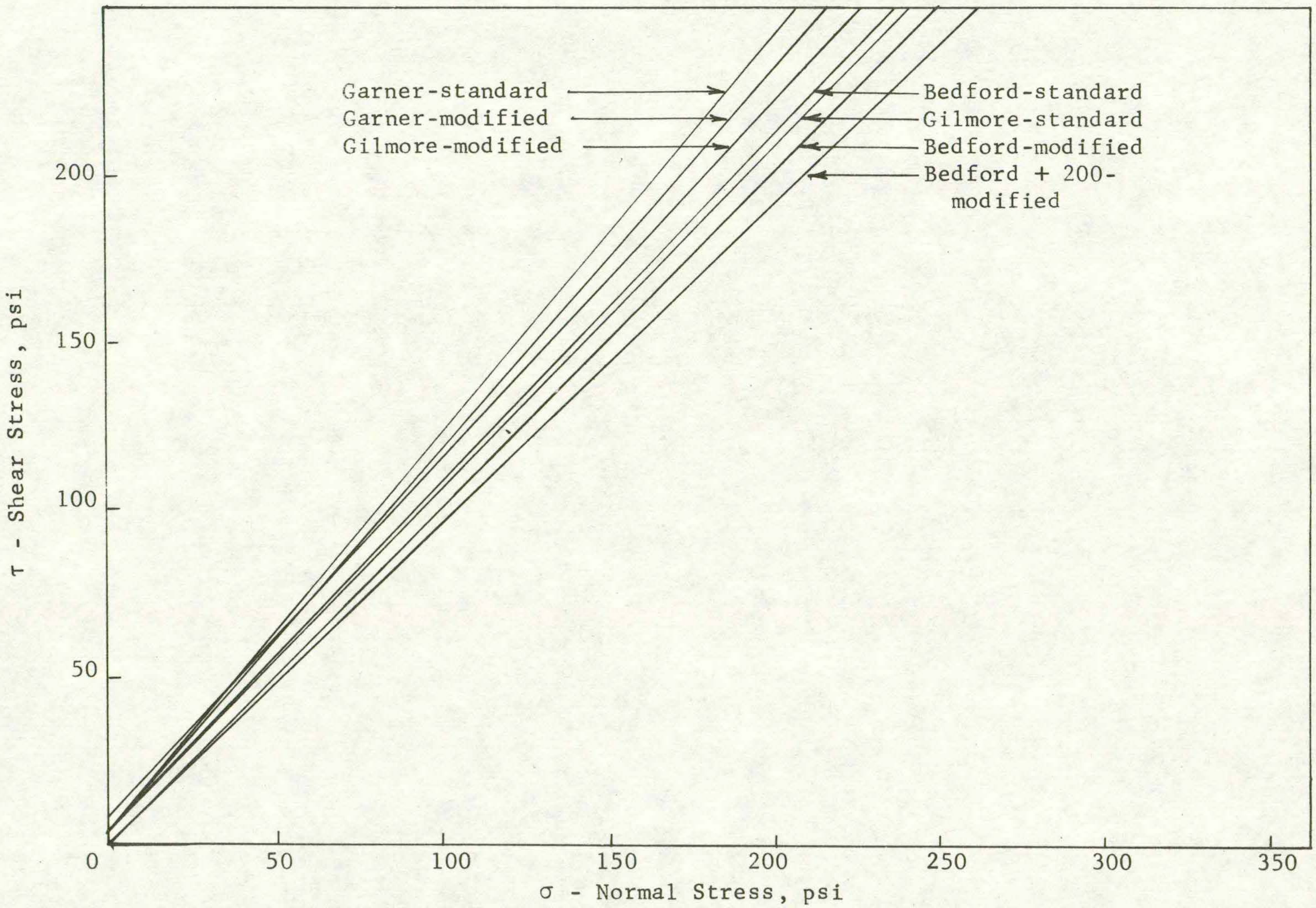

Figure 6. Mohr-Coulomb envelopes for the crushed stones investigated at the point of minimum volume

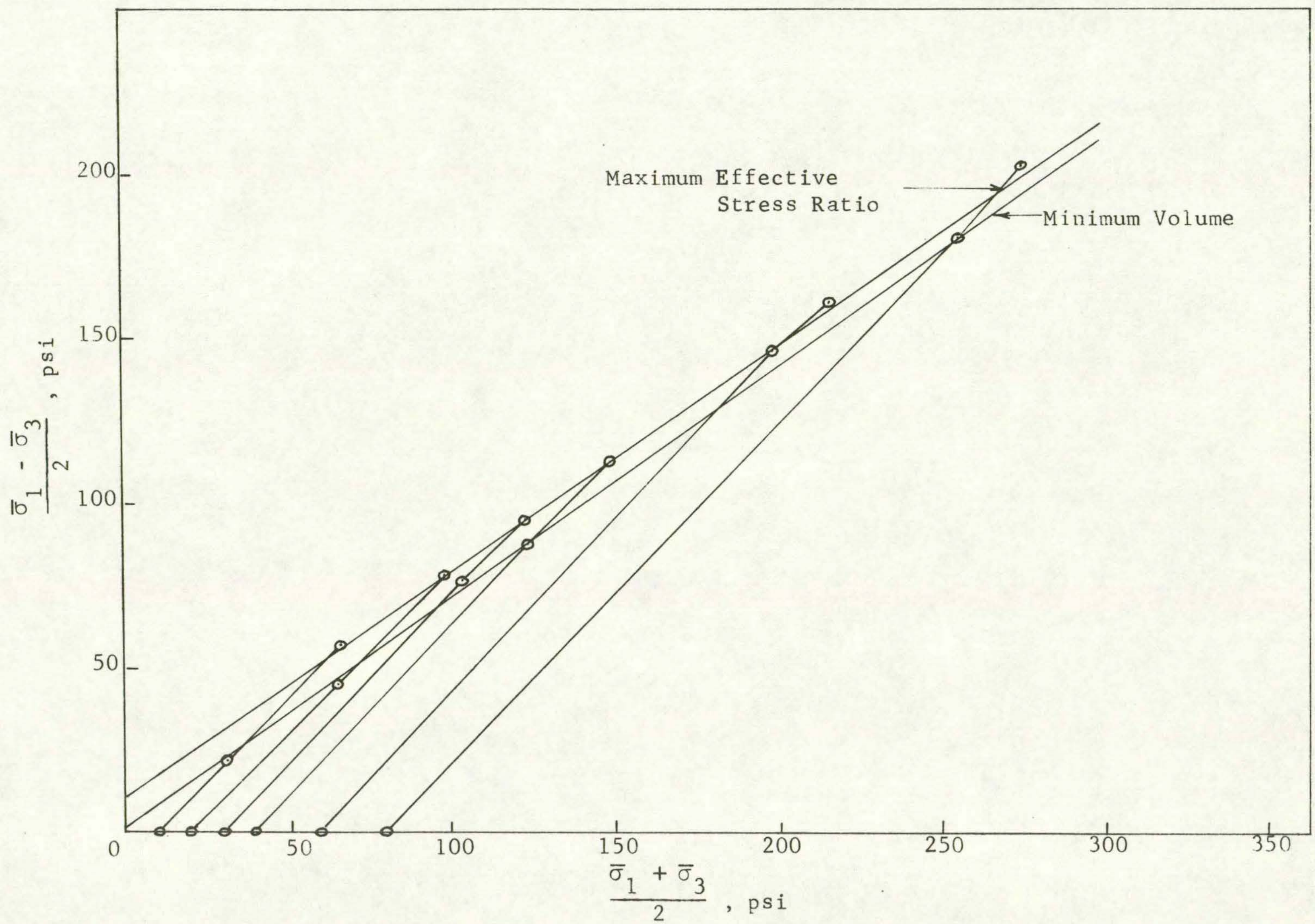

Figure 7. "Stress path" for Bedford at modified Proctor density with envelopes for minimum volume and maximum effective stress ratio failure criterion.

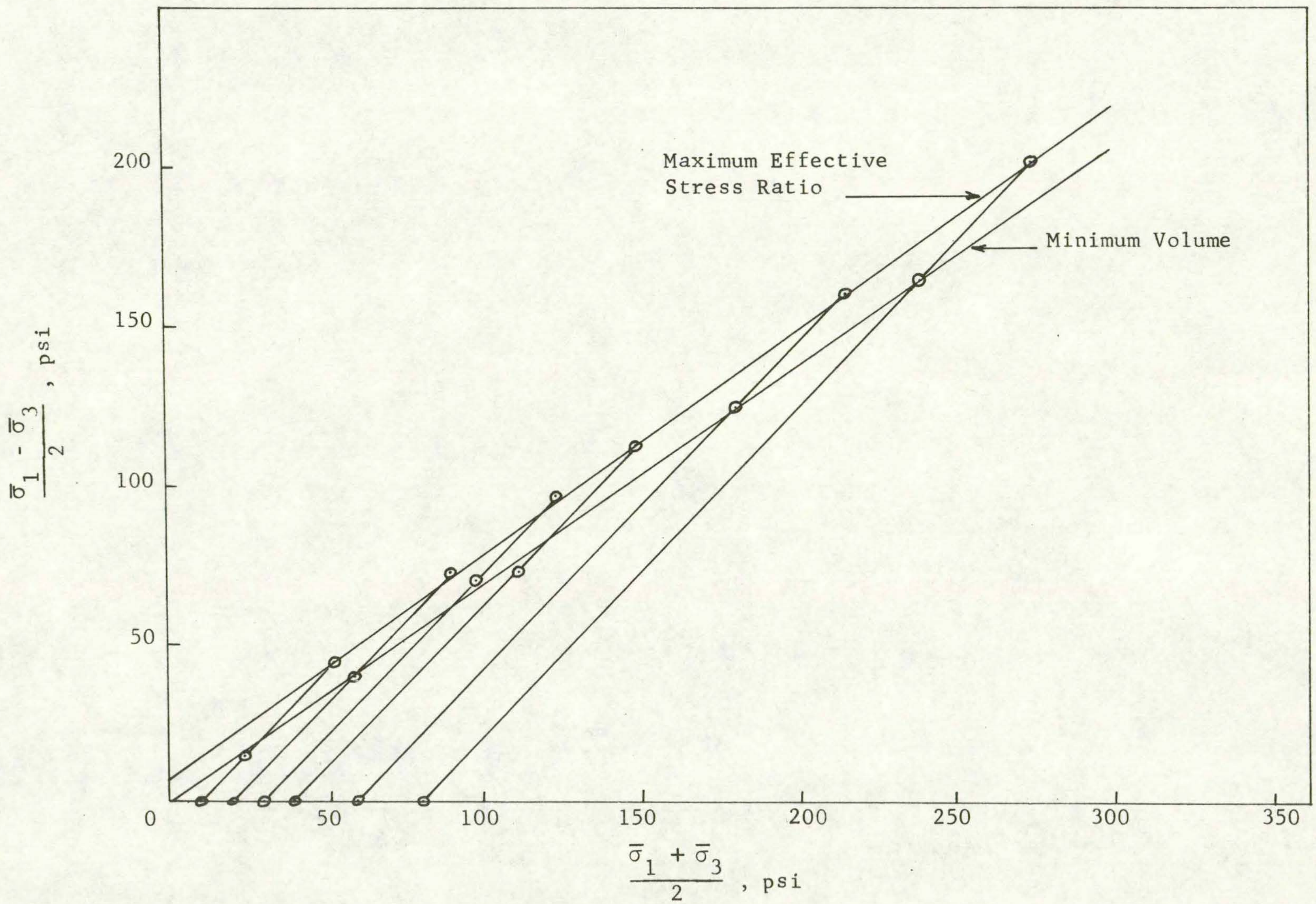

Figure 8. "Stress path" for Bedford + 200 at modified Proctor density with envelopes for minimum volume and maximum effective stress ratio failure criterion

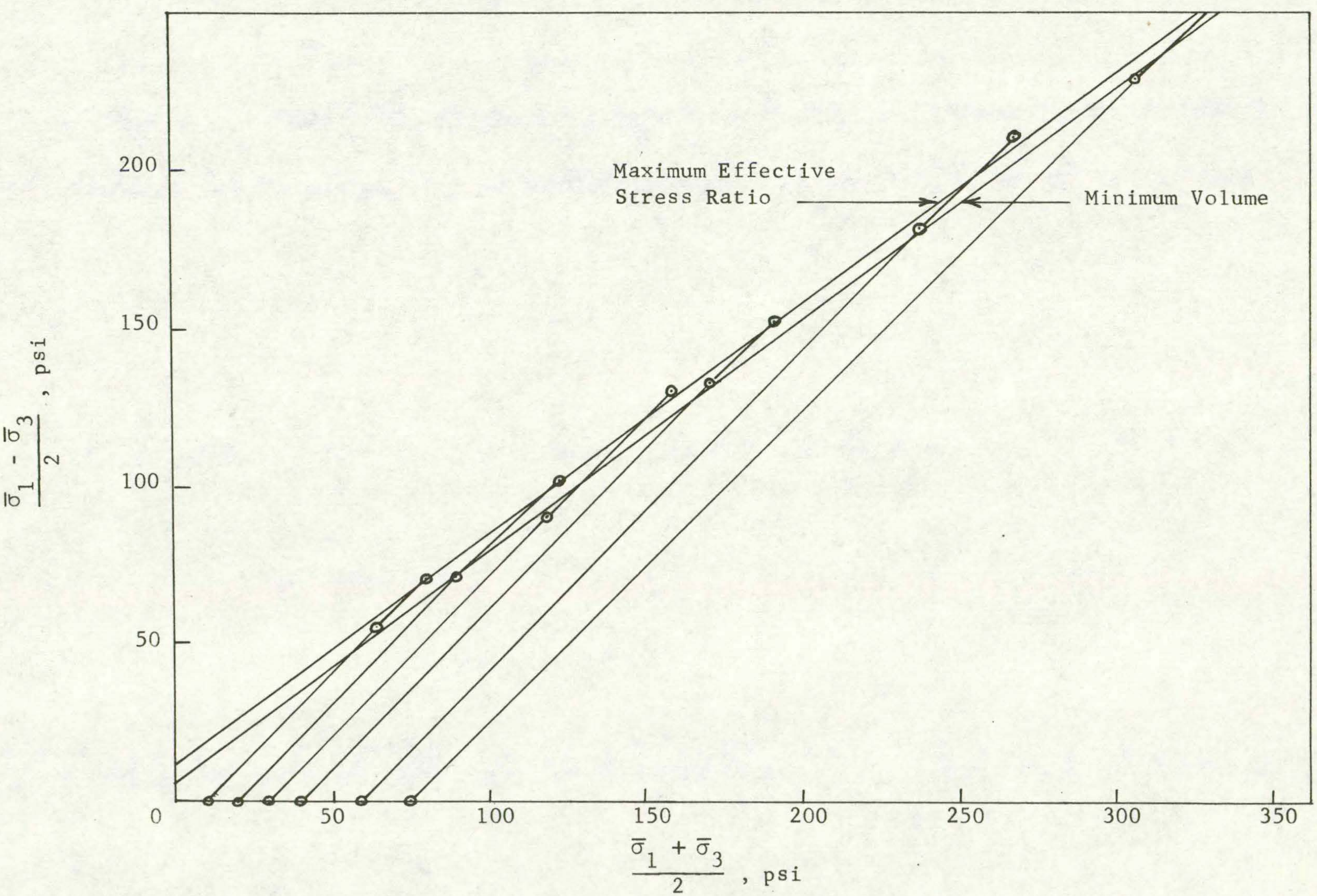

Figure 9. "Stress path" for Gilmore at modified Proctor density with envelopes for minimum volume and maximum effective stress ratio failure criterion

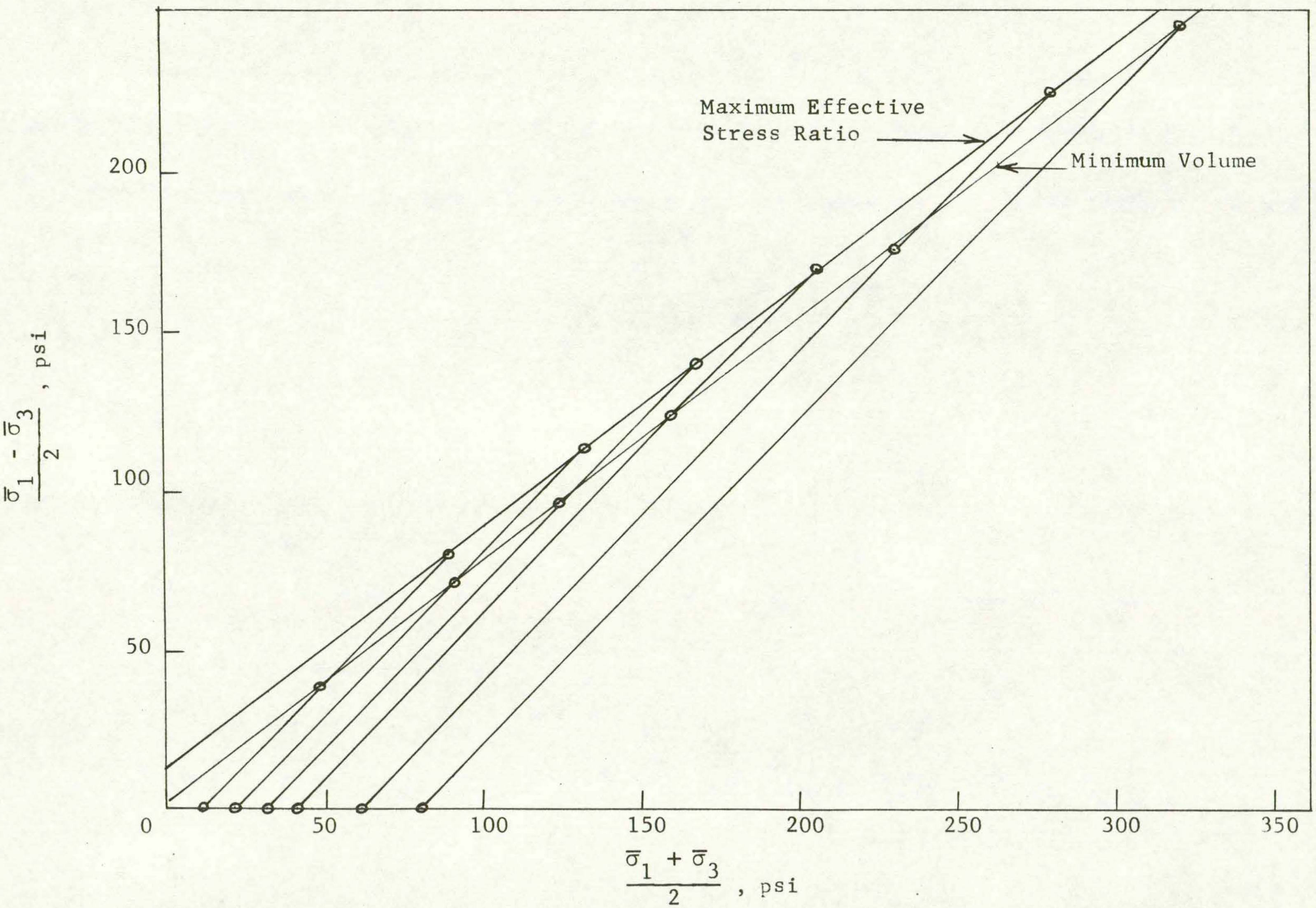

Figure 10. "Stress path" for Garner at modified Proctor density with envelopes for minimum volume and maximum effective stress ratio failure criterion

error and/or slight changes in particle shape produced by the compaction to a higher density.

Increasing the density resulted in a lower cohesion at minimum volume, again reflecting particle interlock. Specimens at standard Proctor density require a higher amount of particle movement to reach minimum volume than specimens at modified Proctor density and though expended this added energy results in a higher cohesion at minimum volume.

The angle of internal friction of the Bedford + 200 sieve materials at modified density was reduced 1.5 degrees from that of the whole Bedford. This difference may be due to normal experimental errors; however, the quantity of fines may be insufficient to completely fill the voids and less frictional resistance is thus developed.

In the whole Bedford, the quantity of fines may be overfilling the voids, causing added frictional resistance. This effect was noted in the investigation by Yoder (35) and the National Crushed Stone Association (24). The latter investigation indicated that shear strength decreased in the order of 9.0, 4.5, 13.0, 1.0, and 20.0 per cent passing the No. 200 sieve, respectively. This indicated that there is an optimum per cent passing the No. 200 sieve for a particular granular material.

With reduction of fines, the cohesion reduced slightly indicating that there was no significant change in the degree of interlocking between the Bedford and the Bedford + 200 when compacted to the same degree of density.

The variance in friction angle for the crushed stones investigated at modified Proctor density was 43.3 to 48.6 degrees. It is felt that this difference is the result of minor variances in the physical properties of the crushed stones; i.e., void ratio, particle shape and size, gradation, surface texture, plasticity of the soil binder and degree of saturation. Many previous investigations (13, 15, 17, 21, 24) have shown that the friction angle may be controlled by these factors.

The cohesion varied significantly between the three stones at modified density; i.e., 1.28 to 8.75 psi. For these granular materials it is believed that the difference in cohesion is a measure of the particle interlock and may be controlled by the void ratio, gradation, and size and shape of the particles. Visual examination of the three stones indicated that the Gilmore has the most angular particles which would increase the interlocking. The Bedford was found to have the highest percentage of degradation in the investigation by Hoover (18). This degradation results in the rounding off of sharp edges, which would result in a lower degree of particle interlock and therefore, lower cohesion at the point of minimum volume in comparison to the other stones.

The results indicate that more than one and possibly all of the aforementioned factors are producing the differences in shear strength for the crushed stones. The void ratio decreases and the shear strength increases in the order of Bedford, Gilmore, and Garner at modified density. The same relationship is true for the stones at standard density but for identical stones a decrease in void ratio results in an increase in shear strength for the Gilmore stone only. The Bedford and Garner standards have a higher void ratio than the Bedford and Garner at modified density, but have higher shear strength without regard to the other factors. Two possibilities exist for this lower shear strength at higher density: a) as mentioned previously, the particle shape has been changed during compaction, or b) there is an optimum density for maximum shear strength, any greater density resulting in a lowering of the shear strength.

As will be discussed in the following section it is thought that the standard specimens have a higher degree of saturation after the consolidation phase of the triaxial shear tests. This should tend to cause lower shear strength in the standard specimens.

The soil binder (passing the No. 40 sieve) of the Garner and Gilmore was non-plastic and that of the Bedford was slightly plastic (PI = 2). Due to the plasticity a slightly lower friction angle should be produced for the Bedford. Removing the minus 200 sieve portion of the soil binder appeared to have little effect on the plasticity and no significant conclusions can be drawn between the plasticity of the soil binder for the Bedford and Bedford + 200.

As indicated by these results, the use of the shear strength parameters ϕ' and C', as a measure of the stability of granular base course mixes, can be misleading. An increase in density has resulted in a lower overall shear strength for the Bedford and Garner which is contrary to the general theory of compaction for granular materials. Other factors to be discussed in later sections are thought to be better indicators of stability of the materials and show a marked improvement with an increase in density.

Strain-Volume Change-Pore Pressure

The relationship of strain and volume change, at the point of minimum volume, and maximum pore pressure, which developed at or near the point of minimum volume, versus applied lateral pressure is given in Figures 11, 12, and 13 for the Bedford, Gilmore, and Garner stones, respectively. The per cent strain necessary to bring the specimens to minimum volume was reduced by an increase in density from standard to modified, with each stone. Generally, the Bedford, Bedford + 200, Gilmore, and Garner at modified density show nearly equal amounts of strain at identical lateral pressures. Similar observations were noted for the per cent volume change at the point of minimum volume over the range of lateral pressures tested.

The above results suggest that all of the stones compacted to modified Proctor density behaved similarly even though there was a relatively large difference in initial modified densities of each of the three. Such results tend to reinforce the concept of relative density as being a satisfactory approach to the compaction of granular materials as each stone in this study was compacted under identical vibratory conditions; i.e., at the same per cent relative density. In addition, the average densities of all specimens compacted to modified Proctor were within 99.3 per cent of AASHO modified density, indicating a uniformity of control during laboratory compaction.

It appears that satisfactory control of densities was not accomplished on the specimens compacted to standard Proctor. The Bedford standard average density was 99.4 per cent of AASHO standard

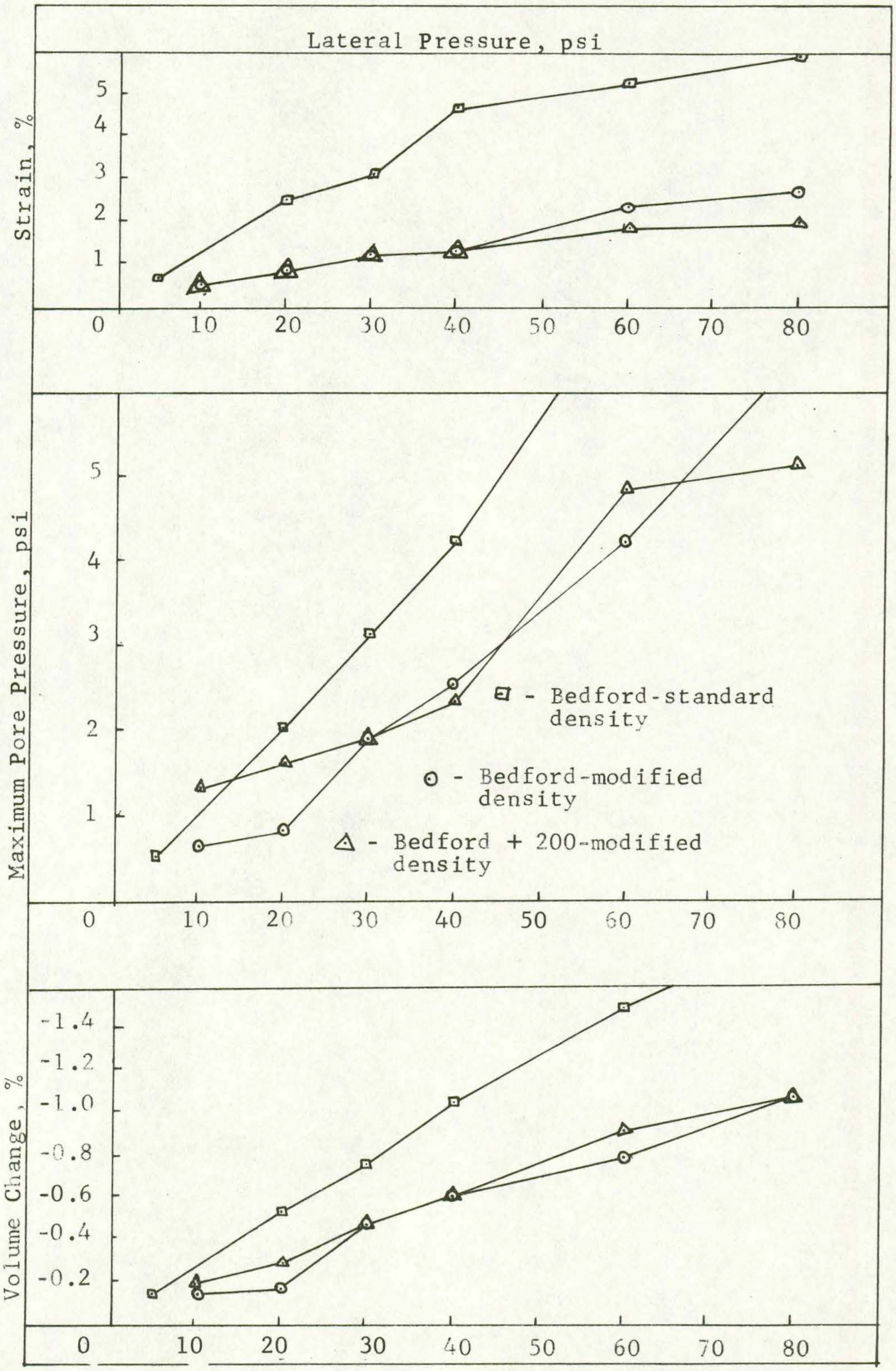

Figure 11. Strain-pore pressure-volume change versus lateral pressure for Bedford stone

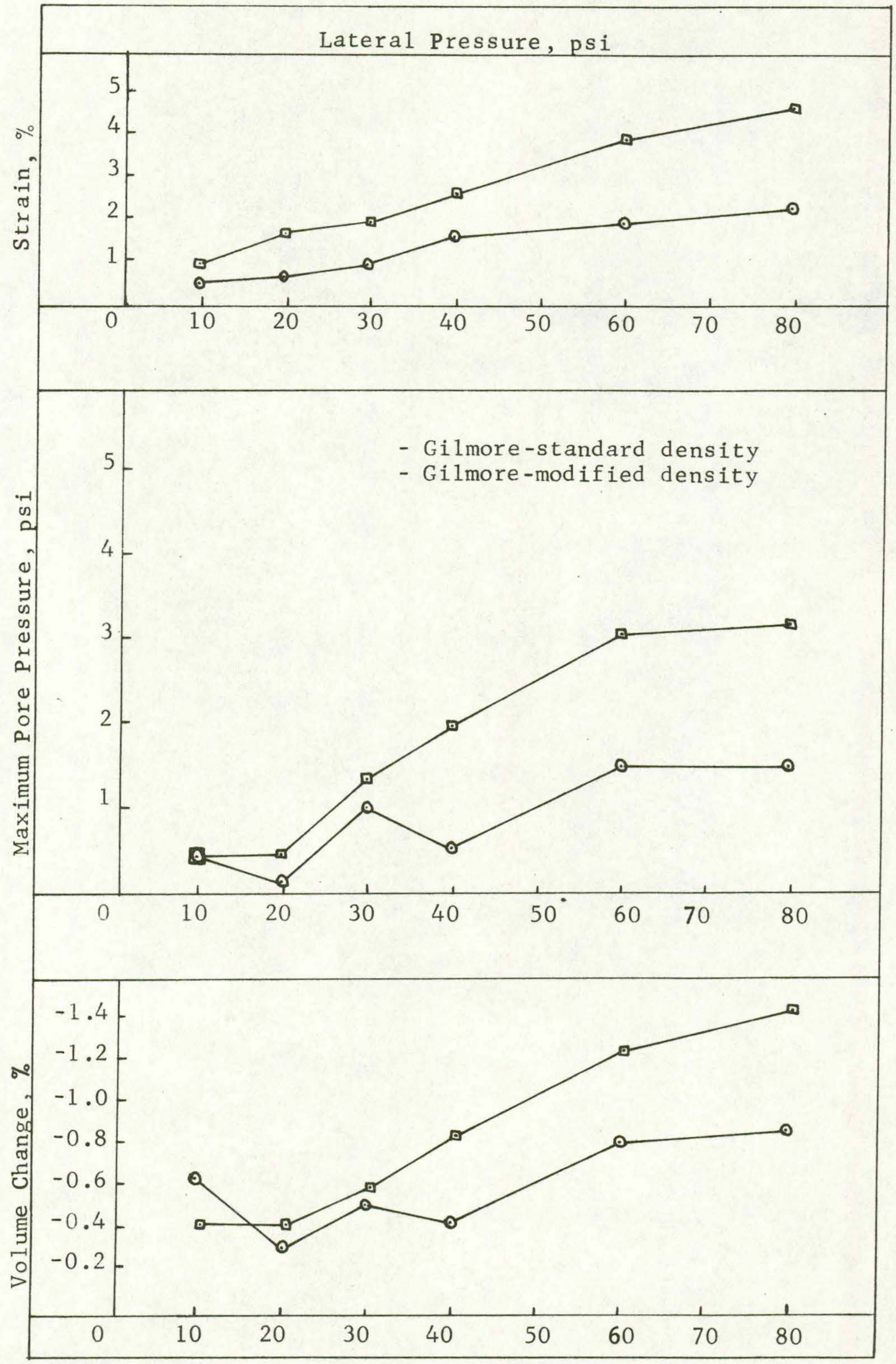

Figure 12. Strain-pore pressure-volume change versus lateral pressure for Gilmore stone

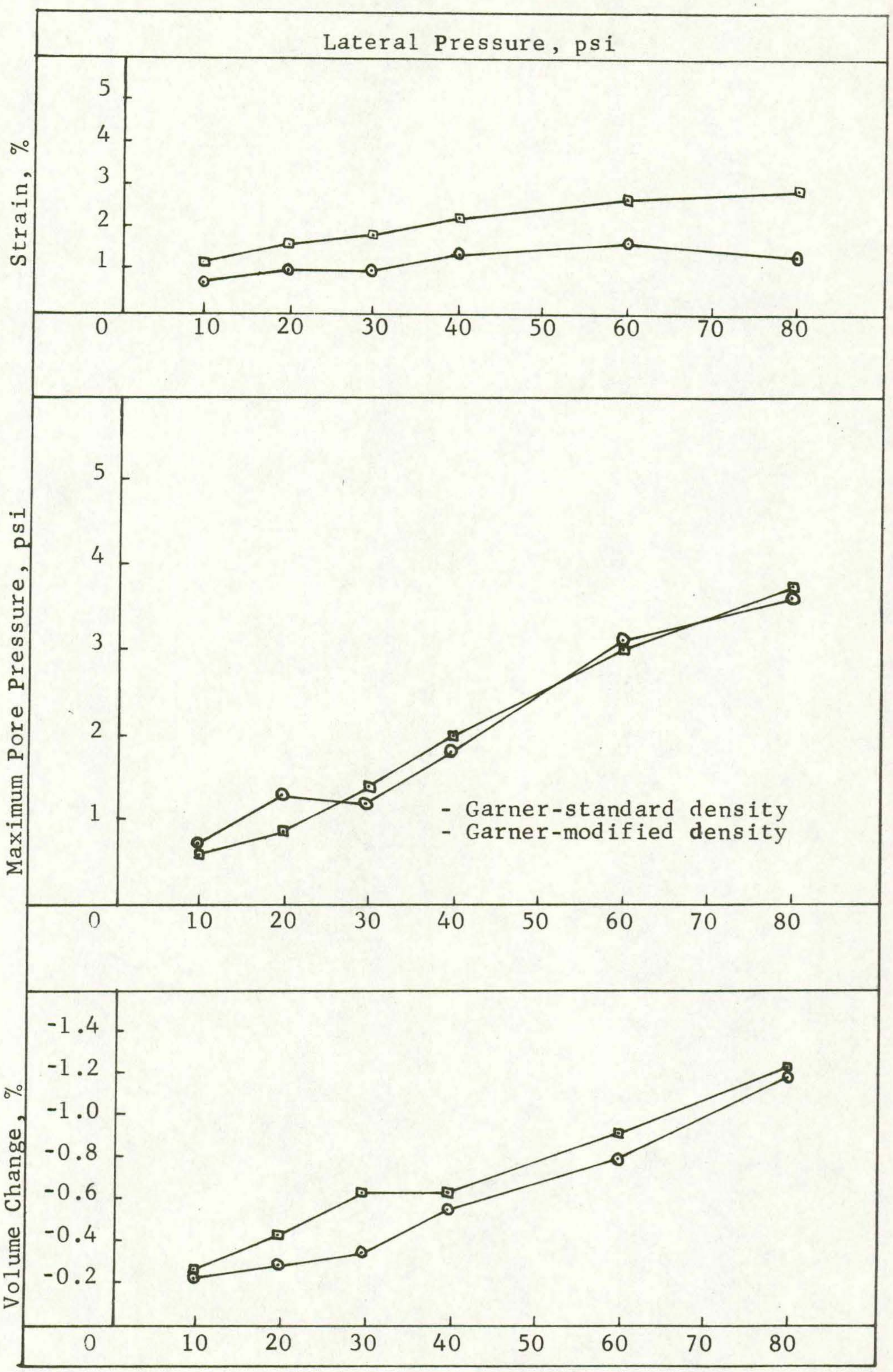

Figure 13. Strain-pore pressure-volume change versus lateral pressure for Garner stone

density, whereas the Gilmore standard was 102.3 per cent and the Garner standard was 103.5 per cent of AASHO standard density. A slight difference of only 1.7 pcf existed between the modified Garner and Garner standard average densities. No attempt is made herein to explain such variations within the standard density specimens. It was previously noted that the materials compacted to modified density showed nearly equal amounts of strain and volume change at identical lateral pressures. Because of the deviations from actual standard density, the specimens at "near" standard Proctor compaction do not show similar characteristics regarding strain and volume change versus lateral pressures. For example, the strain and volume change at 60 psi lateral pressure of the Bedford standard is nearly twice that of the Garner standard, while the Gilmore standard lies in between. The results noted however, appear as a logical progression as the density of the specimens approach modified Proctor, but also indicate that the "standard" compaction specimens were not at the same degree of relative density. It will be noted that for the modified Garner and Garner standard, the per cent volume change is nearly equal.

In undrained triaxial shear tests on saturated specimens there should be no volume change. The tendency of specimens to change volume may then be measured by pore pressure, positive pressures indicating a tendency to reduce volume and negative pressures indicating a tendency to increase volume. When specimens are not completely saturated in undrained tests, both volume change and pore pressures exist, due to compressible void air permitting volume change to occur. This condition produces a reduction in the magnitude of the pore pressures. The lower the degree of saturation, the more reduction in the pore pressures.

Maximum positive pore pressures for the various lateral pressures with each stone are shown in Figures 11, 12, and 13. It will be noted that increased density reduced maximum pore pressures at the given lateral pressures. This reduction appears directly proportional to the change in density between modified and standard of each stone. For the Garner specimens with the small difference in densities, the pore pressures were not significantly reduced. The reduction is greater for the Gilmore and Bedford stones since there is a larger difference in initial densities from the standard to the modified specimens.

Similar results were also found in the investigation conducted by the Melbourne and Metropolitan Board of Works (12). Also the slope of their pore pressure versus total stress curve was lowered by decreasing the degree of saturation.

The degree of saturation that existed in the compacted crushed stones following the consolidation phase of the triaxial shear tests is unknown. It is thought that the specimens at standard Proctor density were at a higher degree of saturation because: a) a higher percentage of volume change occurred with the standard specimens, and b) the initial moisture content was greater.

If the degree of saturation and the volume change were identical for all specimens, regardless of density, then the slope of the pore pressure versus lateral pressure curves of Figures 11, 12, and 13 would be the same. This condition was shown for the Garner modified and Garner standard specimens. However, the two factors of lower percentage of volume change and degree of saturation at minimum volume resulted in lower pore pressures for the Gilmore and Bedford specimens compacted to modified Proctor density as compared with the standard compaction specimens of the two stones.

At modified density, removal of the fines from the Bedford materials did not result in significant change of pore pressure in comparison to the whole Bedford at any of the lateral pressures.

As noted previously, the per cent volume change at minimum volume was generally equal for the stones at modified Proctor density. Thus, the difference in maximum pore pressure can be interpreted to indicate differences in the degree of saturation for each of the stones. The Bedford and Bedford + 200 having the highest degree of saturation and the Gilmore having the lowest for the specimens at modified density.

Comparison of the magnitude of pore pressures of the three stones, Figures 11, 12, and 13, under the two compactive conditions appears to be appropriate. Values of pore pressure of the Garner and Gilmore materials do not appear to be significantly high and are probably of little consequence in base course usage as long as capillary conditions are nonconducive to moisture flow into these materials. However, at lateral pressure conditions of 60 and 80 psi, pore pressures were of the order of 4.5 and 6.2, and 7.2 and 10.0 psi for modified and standard densities, respectively. Stated another way, pore pressures thus ranged from about 650 psf to near 1500 psf at 60 and 80 psi lateral pressures with the Bedford; i.e., adequately high to be of considerable significance in the overall stability of base courses constructed of similar materials. The necessity of preventing high degrees of saturation in base courses is obvious.

Stress-Strain

Deviator stress ($\overline{\sigma}_1$ - $\overline{\sigma}_3$)-axial strain curves resulting from the triaxial tests on the crushed stones investigated at modified density are presented in Figures 14, 15, 16, and 17. Comparison of these figures show a progression under increasing lateral pressure and density, from curves that are typical of loose materials to curves that are typical of dense materials; i.e., respectively, deviator stress increases to a maximum and then remains constant with increasing strain, to deviator stress increases to a maximum and then decreases with increasing strain.

The strain required to produce maximum deviator stress appears relatively independent of the applied lateral pressure, but is dependent on the type of stone. This value of strain for specimens compacted to modified density is approximately 3 per cent for the Garner, 4 per cent for the Gilmore, and 5 per cent for the Bedford and Bedford + 200. Corresponding values for standard Proctor density specimens are approximately 4 per cent for the Garner standard, 5 per cent for the Gilmore standard, and 6 per cent for the Bedford standard. A marked difference is also indicated in the maximum deviator stress for the different stones at each lateral pressure. For example, at 80 psi lateral pressure, the maximum deviator stress at modified is 565 psi for the Garner, 513 psi for Gilmore, 403 psi for Bedford, and 401 psi for Bedford + 200. The difference in deviator stress between identical stones at different densities was not as great. Values for the stones at standard Proctor density are Garner standard 542 psi, Gilmore standard 462 psi, and Bedford standard 389 psi.

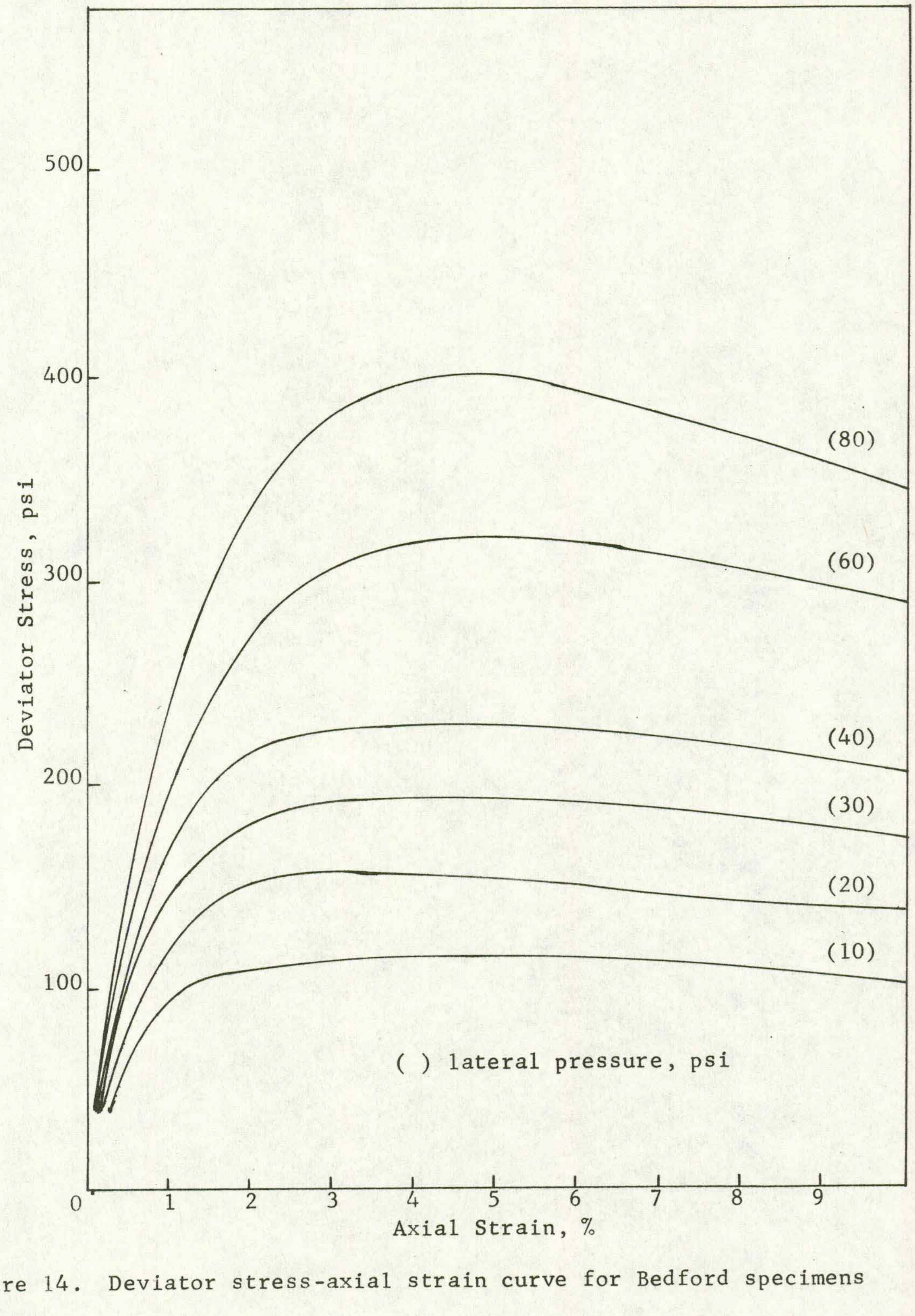

Figure 14. Deviator stress-axial strain curve for Bedford specimens

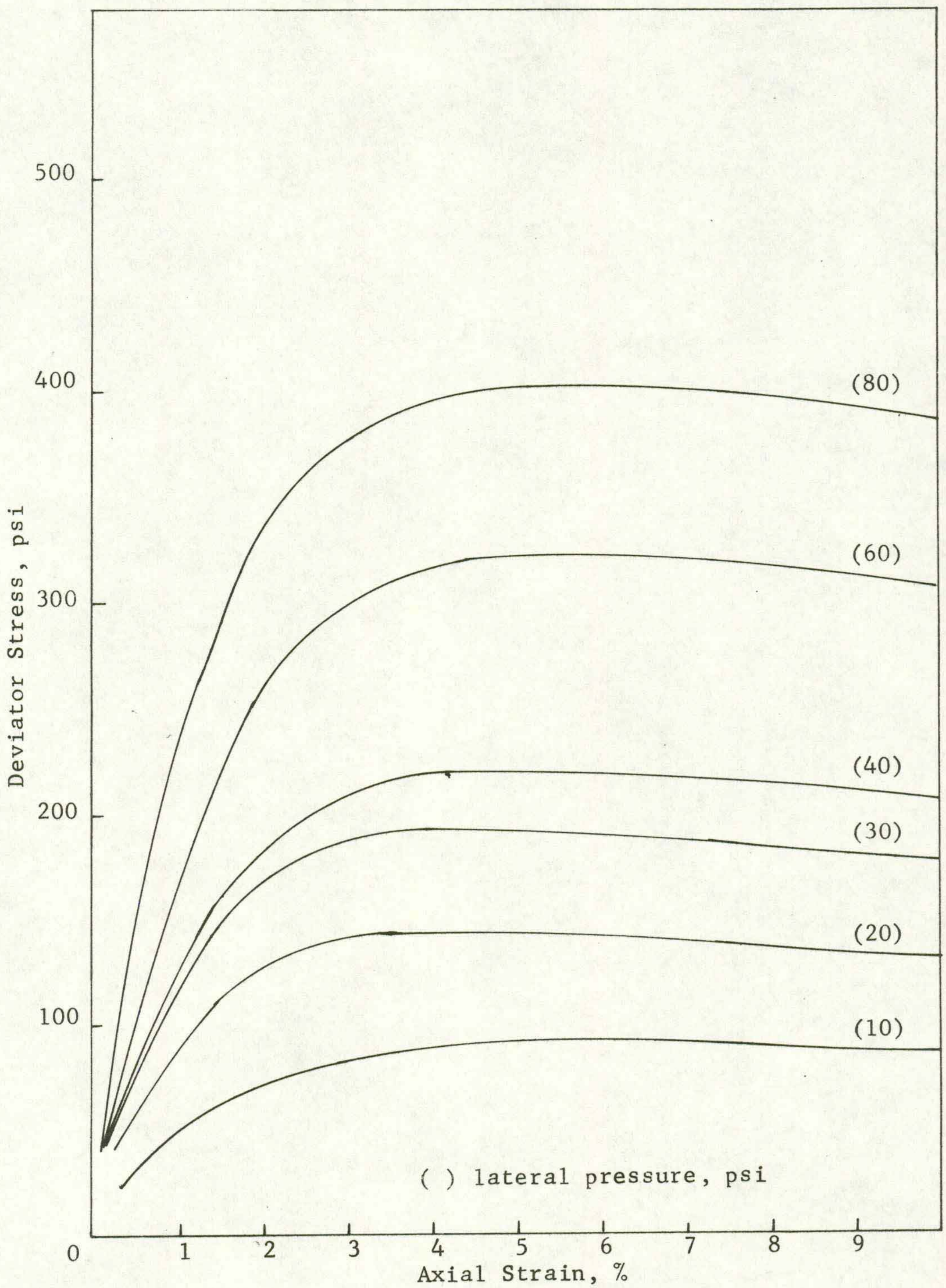

Figure 15. Deviator stress-axial strain curve for Bedford + 200 specimens

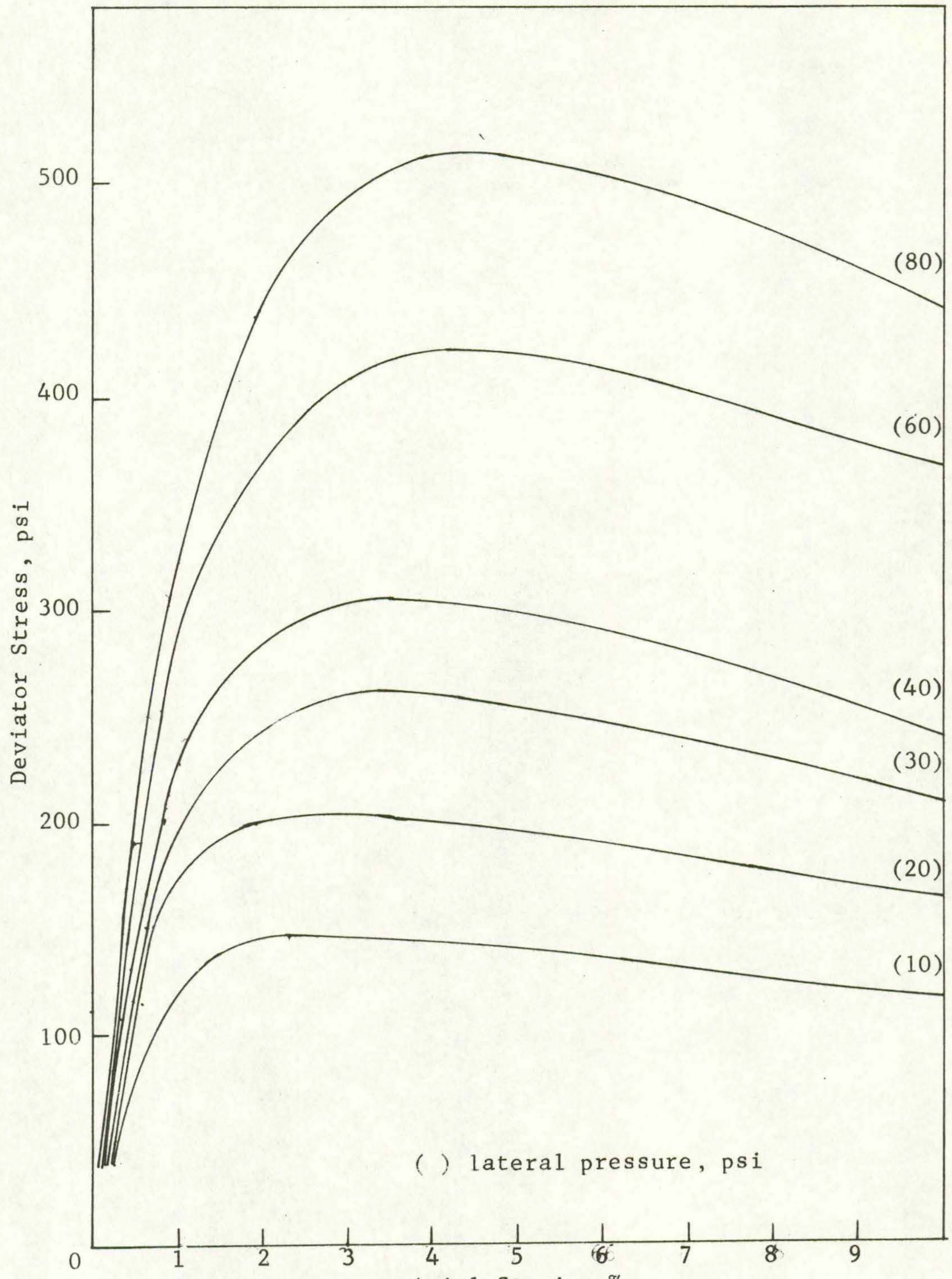

Figure 16. Deviator stress-axial strain curve for Gilmore specimens

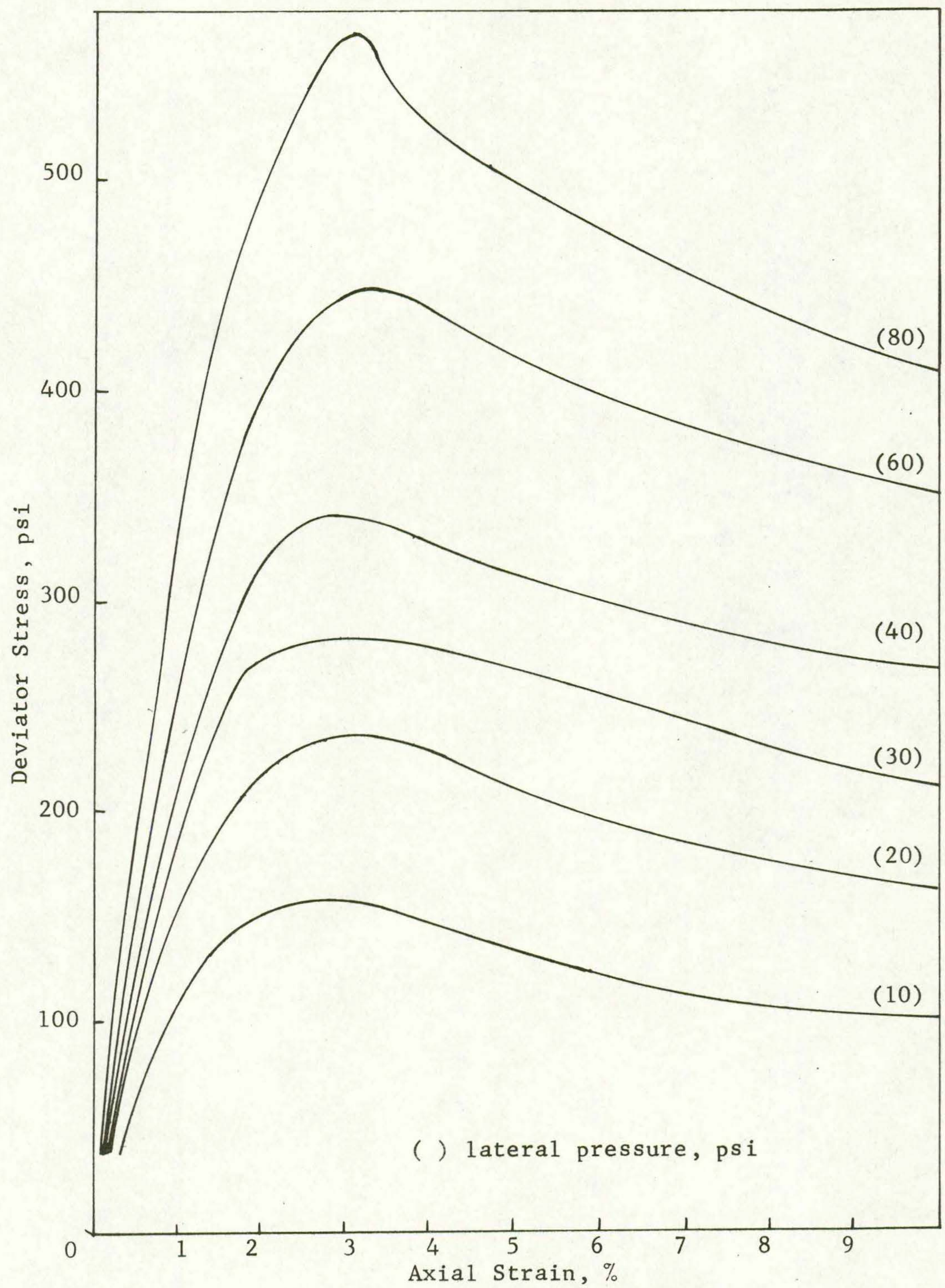

Figure 17. Deviator stress-axial strain curves for Garner specimens

Figure 18 shows stress-strain curves for the modified Bedford, Bedford + 200, and Bedford standard at 30 psi lateral pressure. It is used to illustrate the effects of density on stress-strain results and is typical of all lateral pressures and stones. As illustrated, increasing the density resulted in an improvement in the stress-strain performance at lower values of strain. However, above the strain producing maximum deviator stress in the standard compacted stones the behavior of the materials is much the same regardless of state of densification.

At modified density, the reduction of fines in the Bedford resulted in poorer stress-strain performance at lower lateral pressure and percentage of strain, but the performance of the two Bedford materials became much alike at higher lateral pressures and strains.

The relationship between stress and strain for soils, unlike other construction materials, does not always follow the mechanics of materials laws governing such relationships. For example, there are many ways to define a so-called "modulus of elasticity" for soils. Some investigators use the slope of the initial straight line portion of the stress-strain curves, others use the stress at a given allowable strain and still others determine the stress that should exist under design conditions and use the corresponding strain (35). The investigators herein have defined a "stress modulus" as the deviator stress ($\bar{\sigma}_1$ - $\bar{\sigma}_3$) that produces the minimum volume, divided by the corresponding strain.

As proposed earlier under Shear Strength Criterion, the failure of granular base courses occurs at or near the point of minimum value cor-

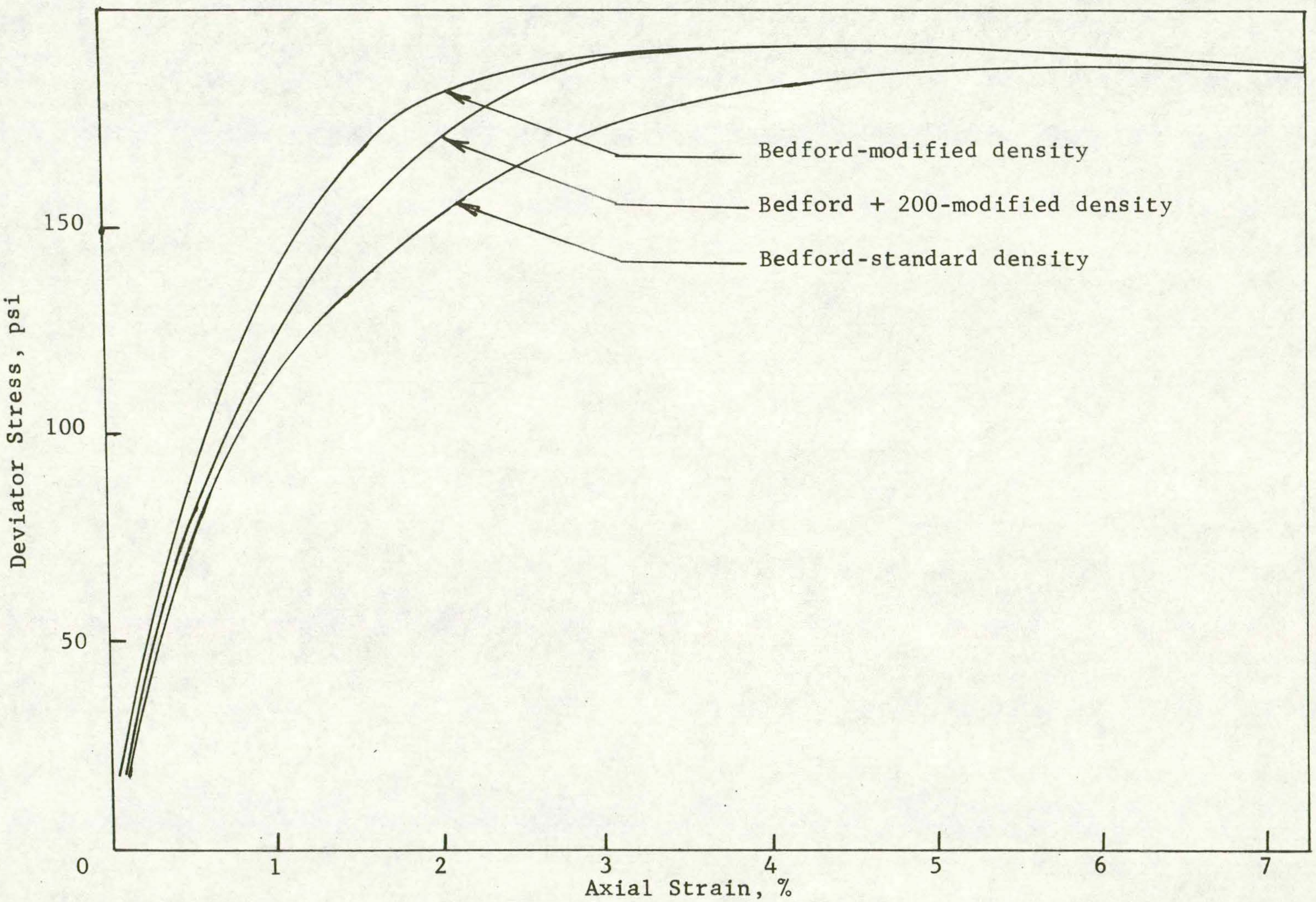

Figure 18. Deviator stress-axial strain curves for Bedford modified, Bedford + 200 modified, and Bedford standard at 300 psi lateral pressure

responding to the lateral pressure that can be developed by the base course. Values of deviator stress versus the strain at minimum volume are plotted in Figure 19. From these values the "stress modulus," as defined above, was computed for each stone at the various lateral pressures tested. Figure 20 shows the trend of the calculated "stress modulus" with increasing lateral pressures for all materials tested in this study plus those tested under standard Proctor density conditions (18). Variations of density previously noted, particularly for the standard compaction specimens, greatly affected this plot and resulted in some scattering of the "stress modulus" values due to the differences in initial densities. Because of this only the trend of the "stress modulus" is given. Indicated are two different behaviors: a) the lateral pressure has no effect on the "stress modulus", and b) the "stress modulus" increases as the lateral pressure increases. It is thought that these behaviors are partially controlled by initial density. The slope of the "stress modulus" versus lateral pressure relationships for the Bedford modified, Bedford standard, Gilmore modified, and Gilmore standard exhibited no basic change over the range of lateral pressures tested. These materials easily decrease to minimum volume with an increase in stress and proportional amount of strain at the different lateral pressures and are the least dense of the stones investigated. The Garner modified and Garner standard are at such a high density, particularly after the consolidation phase of the triaxial test, that the stress to produce minimum volume is not proportional to to the strain as with the other materials. This is shown in Figure 19. In the Garner modified and Garner standard the stress-strain stability

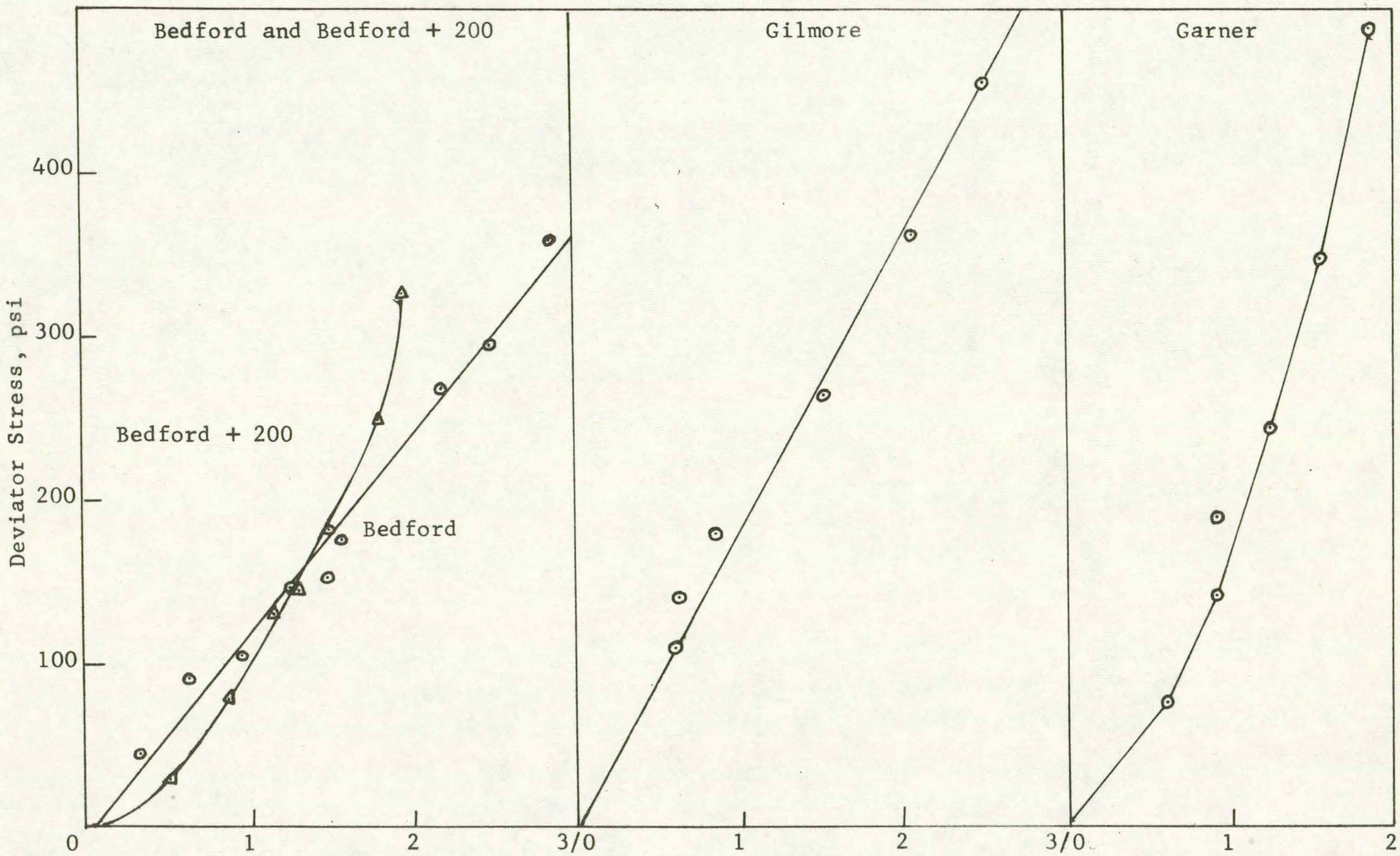

Figure 19. Deviator stress-axial strain relationship at minimum volume for the crushed stones at modified Proctor density

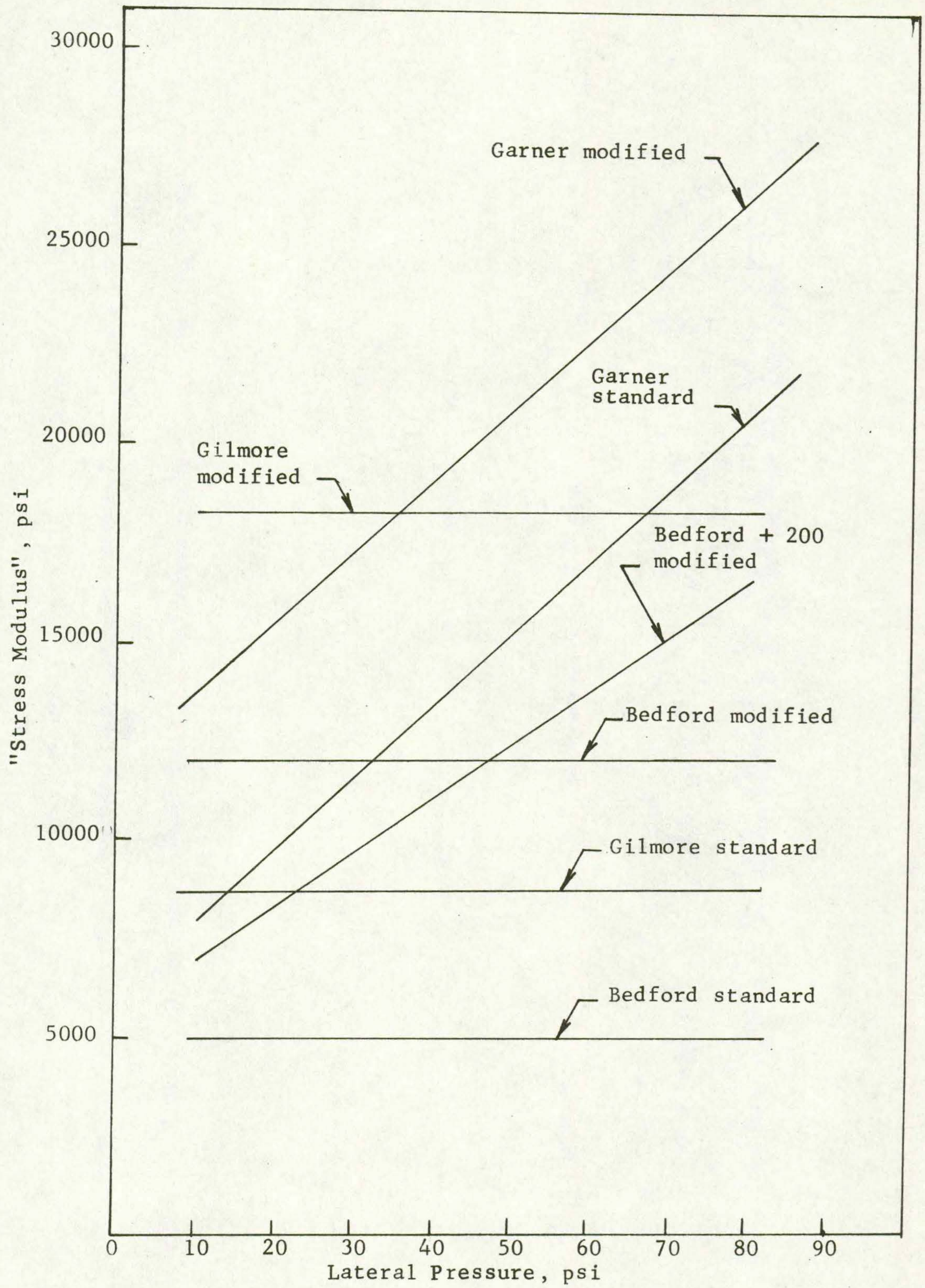

Figure 20. "Stress modulus" at minimum volume versus lateral pressure for the crushed stones

is thus increased with increasing lateral pressures.

Removal of the fines from the Bedford (Bedford + 200) resulted in a significant change in the "stress modulus"-lateral pressure relationship even though the densities were equivalent. The Bedford + 200 shows the same trend as the Garner in Figure 20. It is thought that the lubrication effect of the fines has been removed and therefore the stress to produce minimum volume is not proportional to the strain over the range of lateral pressures tested. This reduction is not beneficial at the lower lateral pressures and only when the lateral pressure is above the 40-50 psi range does the modified Bedford + 200 perform better than the Bedford at modified density.

It is felt that the "stress modulus"-lateral pressure relationship provides a reasonably good measure of the stones' stability. The steeper the slope, the more stable the stones as demonstrated by the Garner versus the Bedford and Gilmore. This is verifiable to a certain degree in that the Garner has the best field service record of the three stones as base course materials. However, it also appears that increases in density may be beneficial at least as measured by the "stress modulus" concept, herein presented, since the trend values of "stress modulus" are increased significantly between standard and modified compaction.

Stress-Volume Change

Figure 21 shows the deviator stress-volume change relationship for a modified density Bedford + 200 specimen with lateral pressure of 60 psi. It is used herein as being typical of the above relationship for all the materials and also demonstrates the mechanism of failure

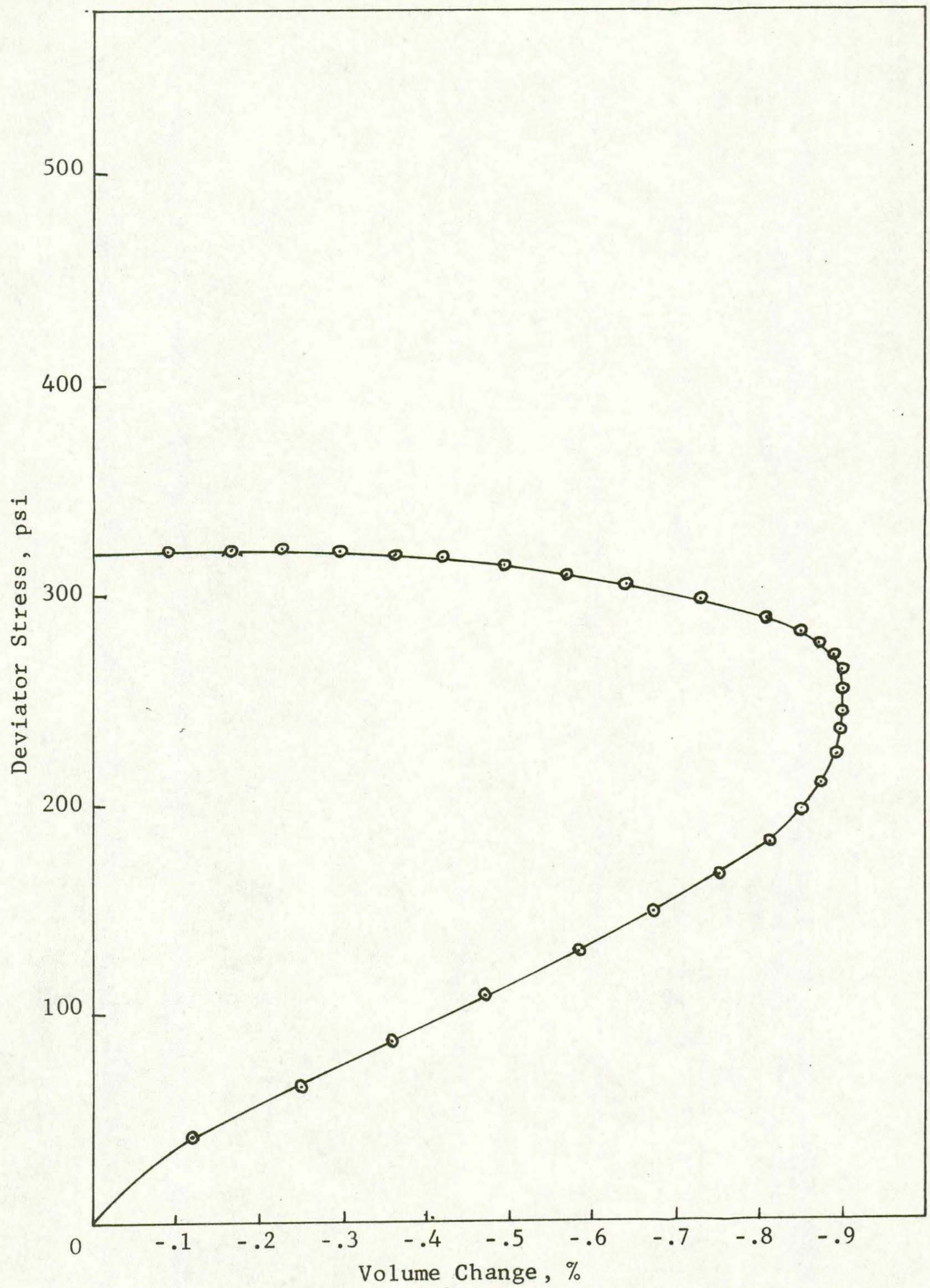

Figure 21. Deviator stress-volume change relationship for Bedford + 200 at 60 psi lateral pressure

discussed previously.

As illustrated by Figure 21, as the deviator stress is increased, the specimen reduces in volume until a minimum volume is reached corresponding to the lateral pressure used. At this point a slight increase in deviator stress results in lateral deformation producing a volume increase. In these laboratory tests the lateral pressure has been held constant which results in a higher deviator stress beyond the point of minimum volume change. In field conditions, the lateral pressure may not remain constant, but may decrease when lateral deformation occurs. Therefore, the field stress-volume change relationship may not exhibit an increase in stress as lateral deformation and expansion occurs, but would tend to decrease due to decreasing lateral pressure.

Figure 22 gives the relationship between the deviator stress and the per cent volume decrease at minimum volume and varying lateral pressure for all materials tested in this and previous studies. The steeper the slope of the deviator stress-volume change line, the more stable the material. The results indicate that the modified Garner, Garner standard, and modified Gilmore behave similarly. An increase in density improved the Bedford and Gilmore stones and produced a lower percentage of volume change under any given stress condition. Removal of the fines from the Bedford was detrimental, in that a lower stress was required to produce an equal percentage of volume change at minimum volume for the two Bedford materials at modified density.

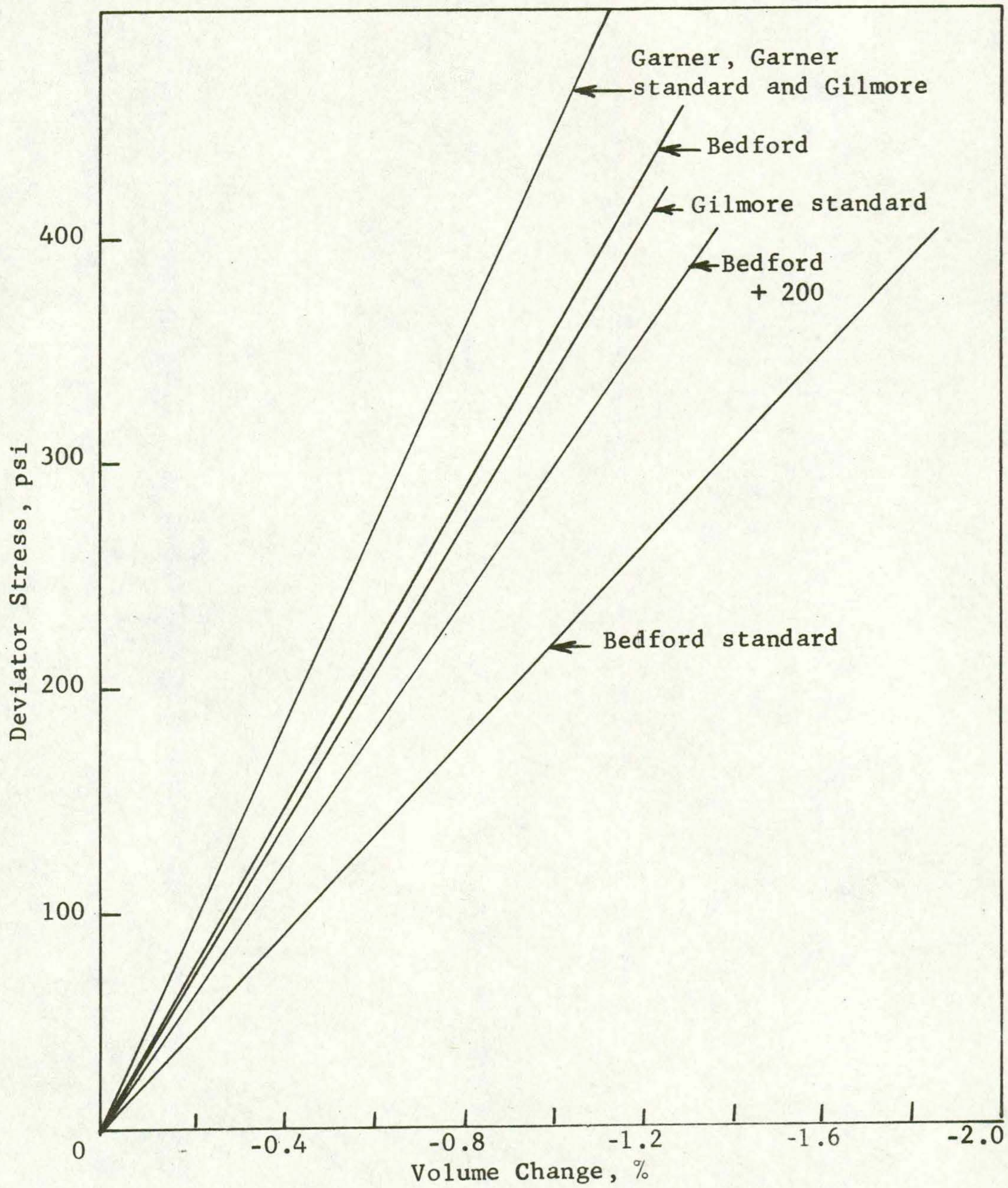

Figure 22. Envelopes of deviator stress-volume change at point of minimum volume for the crushed stones

FINAL DISCUSSION

The three crushed stones were analyzed with respect to the shear strength, stress-strain, and stress-volume change relationships resulting from the triaxial shear tests. Shear strength results at maximum effective stress ratio indicated an extremely high value for cohesion (over 18 psi for the Garner and Gilmore specimens at modified density) in these "cohesionless" soils. Also, the stress-volume change relationship (as in Figure 21) indicated that the point of maximum effective stress ratio was not the critical condition for stability of the materials studied and led to the adoption of the minimum volume condition to define a failure criterion for these granular materials.

The difference in shear strength between the point of minimum volume and maximum effective stress ratio was observed to be mainly an increase in the cohesion parameter and was of the order of about 10 psi or more for all except the Garner; this larger cohesion resulting during expansion of the tested specimens.

A cohesion value still exists at the minimum volume condition due to the interlocking of particles which must be overcome for development of the friction parameter. The results indicate that interlocking is greatest in the Gilmore and least in the Bedford.

Differences existed in the angle of internal friction for the three stones, with Garner having the largest angle and Bedford the smallest. This difference seemed to be due mainly to the difference in density of the stones compacted to modified Proctor. Utilizing the shear strength parameters to determine the overall shear strength stability shows that the Garner has the highest, Gilmore next, and Bedford the lowest.

When comparison is made of the same stone at different density, other factors, such as changes in particle shape and degree of saturation, have a more pronounced effect on the overall shear strength and an increase in density resulted in an increase in shear strength for the Gilmore only.

Removal of the minus 200 sieve portion of the Bedford reduced the friction angle and overall shear strength, probably due to insufficient fines to fill the voids.

Failure of base courses occurs when vertical deflection becomes excessive or when there is lateral movement from under the loaded area. These conditions can be represented by the percentage of axial strain and volume change that occurs during triaxial testing with the minimum volume condition defining the failure point.

For specimens of the crushed stones compacted to modified Proctor density the strain and volume change at minimum volume were about equal for identical lateral pressures. The stress required to produce this strain and volume change differed, resulting in different stability for the stones. This can be shown best by consideration of the "stress modulus" given in Figure 20. At a lateral pressure of 60 psi (which is thought to be a reasonable value for base courses) the "stress modulus" for Garner is 22,500 psi, Gilmore is 18,000 psi, and Bedford is 12,000 psi. If the allowable vertical deflection was 1 per cent (i.e., 0.1 inch for a 10 inch base) then the maximum load would be 285, 240, and 180 psi, respectively for the stones; thus, a great difference in the stability.

The benefit of increased density can be shown by comparison of the 60 psi confining pressure specimens at standard Proctor, where the "stress modulus" is 17,000 psi for Garner, 8,500 psi for Gilmore, and 5,000 psi for Bedford. Corresponding loads under the above conditions would then be 230, 145, and 110 psi.

If the lateral pressure that could be developed in the base was only 20 psi, the stones at modified density would all have reached the minimum volume condition and be in the process of expanding, producing ruts and upheaval of the surrounding material, upon application of any of the maximum loads indicated above. The same would be happening in the Garner standard. The Bedford standard and Gilmore standard would not be brought to the minimum volume condition by the above loads due to their higher requirement of vertical deflection, i.e., of the order of 2 per cent strain. These results indicate that the lateral pressure developed in base courses has a direct effect on the stability under a given set of conditions.

CONCLUSIONS

From analysis of the results obtained during this investigation of the three granular materials it may be concluded that:

1. Minimum volume condition appears as a more valid failure criterion for granular base course mixes than maximum effective stress ratio or maximum deviator stress.
2. Use of shear strength only may not be an entirely acceptable criterion for the determination of stability of granular materials.
3. Stability of granular materials should be evaluated in terms of the shear strength, stress-strain, and stress-volume change relationship.
4. The stability of the crushed stones investigated appears to vary with the applied lateral pressure and initial state of densification.
5. Failure of granular base course materials appears to be controlled by the lateral pressure that can be developed within the base course regions.
6. The increase in density from standard to modified Proctor results in decreased strain, volume change, and pore pressures, improving the stability of the crushed stones. This increase in density has not significantly changed the shear strength or maximum stress that can be sustained by these crushed stones.
7. Pore pressures become significant in granular materials that are at a high degree of saturation.

8. The Garner stone appears to have the greatest stability as measured by shear strength, stress-strain, and stress-volume change. The Bedford stone appears to have the lowest stability as measured by the same relationship.
9. Removal by dry sieving of the minus No. 200 sieve portion of the Bedford stone did not significantly reduce its overall stability, but indicated that there may be an optimum percentage of materials passing the No. 200 sieve.

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ACKNOWLEDGEMENTS

This research is part of a study of the factors influencing stability of granular base course mixes conducted at the Engineering Research Institute at Iowa State University, under sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission, and Bureau of Public Roads, U. S. Department of Commerce.

The authors wish to express their appreciation to Dr. Richard L. Handy, Professor of Civil Engineering, for his council and guidance during the investigation and preparation of this manuscript. Special thanks are due the staff of the Soils Research Laboratory, Engineering Research Institute, for their unselfish assistance during the study.

APPENDIX

Table 7. Summary of triaxial test data for Bedford specimens compacted to modified Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content, %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[a]
BB-1	10	131.5	98.5	9.18	85.5	.293	.275
BB-2	10	133.0	99.6	9.05	87.7	.282	.258
BB-3	20	132.3	99.1	9.02	85.4	.288	.274
BB-4	20	132.4	99.2	9.43	89.5	.288	.260
BB-5	30	131.4	98.4	9.04	83.0	.297	.265
BB-6	30	132.8	99.5	9.00	87.3	.285	.254
BB-7	40	132.3	99.1	9.48	89.6	.289	.248
BB-8	40	133.7	100.2	8.89	88.4	.275	.228
BB-9	60	131.9	98.8	8.91	83.5	.292	.236
BB-10	60	133.4	99.9	9.02	89.1	.276	.221
BB-11	80	133.1	99.7	8.90	86.6	.281	.218
BB-12	80	132.6	99.3	9.18	88.4	.284	.234
Average		132.5	99.3	9.09	87.0		

[a]A. C.: after consolidation.

Table 7. (Continued)

	Minimum Volume Condition					Maximum Effective Stress Ratio				
Specimen Number	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change,%	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %
BB-1	.274	54.6	9.6	0.31	-0.067	.292	122.2	9.3	3.41	+1.280
BB-2	.255	68.7	9.2	0.95	-0.240	.295	105.2	9.6	3.16	+0.810
BB-3	.273	109.9	19.7	0.62	-0.063	.283	176.8	20.5	2.80	+0.679
BB-4	.256	124.0	18.9	0.94	-0.267	.262	168.6	18.8	2.18	+0.187
BB-5	.261	180.9	28.0	1.23	-0.354	.268	218.6	28.4	3.08	+0.228
BB-6	.248	175.1	28.7	1.25	-0.520	.254	218.8	28.5	3.13	-0.040
BB-7	.241	221.0	39.2	1.24	-0.605	.248	273.4	38.9	4.02	+0.012
BB-8	.221	211.5	37.0	1.56	-0.585	.228	263.2	36.3	4.35	-0.024
BB-9	.226	323.2	56.3	2.18	-0.808	.229	352.0	57.2	3.43	-0.629
BB-10	.211	350.1	55.9	2.48	-0.747	.217	376.5	55.3	4.35	-0.268
BB-11	.205	432.4	74.0	2.86	-1.037	.208	457.7	73.5	4.77	-0.765
BB-12	.211	437.1	74.7	2.48	-1.065	.215	475.8	74.0	4.35	-0.691

Table 8. Summary of triaxial test data for Bedford + 200 specimens compacted to modified Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content, %	Initial Saturation, %	Void Ratio Initial	A. C.[a]
BB-201	10	132.2	99.0	9.32	88.0	.289	.268
BB-202	20	132.2	99.0	9.13	86.2	.289	.260
BB-203	30	132.8	99.5	9.00	86.8	.283	.237
BB-204	40	131.9	98.8	9.40	87.9	.292	.250
BB-205	60	134.6	100.8	9.24	94.8	.266	.213
BB-206	80	132.5	99.3	9.08	86.7	.286	.228
Average		132.7	99.4	9.20	88.4		

[a] A. C.: after consolidation.

Table 8. (Continued)

	Minimum Volume Condition					Maximum Effective Stress Ratio				
Specimen Number	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %
BB-201	.266	40.4	9.3	0.57	-0.177	.282	95.9	9.0	4.30	+1.100
BB-202	.257	96.9	18.4	0.89	-0.283	.267	162.3	18.2	3.67	+0.558
BB-203	.231	167.6	28.3	1.33	-0.461	.238	220.2	28.2	3.56	+0.082
BB-204	.242	183.4	38.4	1.31	-0.597	.249	258.1	37.9	4.25	-0.073
BB-205	.202	304.0	56.4	1.81	-0.898	.208	374.8	55.3	4.42	-0.347
BB-206	.215	402.9	76.1	1.97	-1.062	.220	475.7	75.0	4.83	-0.647

Table 9. Summary of triaxial test data for Gilmore specimens compacted to modified Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[a]
BGI-1	10	139.8	99.6	5.33	63.1	.233	.226
BGI-2	20	138.2	98.5	5.92	66.3	.247	.217
BGI-3	30	138.9	99.0	5.87	67.5	.240	.210
BGI-4	40	140.0	99.8	5.70	68.4	.230	.197
BGI-5	60	140.6	100.2	5.87	72.0	.225	.183
BGI-6	80	140.0	99.8	5.75	69.3	.229	.177
Average		139.6	99.5	5.74	67.7		

[a]A. C.: after consolidation.

Table 9. (Continued)

Specimen Number	Minimum Volume Condition: Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %	Maximum Effective Stress Ratio: Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %
BGI-1	.220	120.7	10.8	0.92	-0.444	.229	151.7	11.3	1.84	+0.262
BGI-2	.213	160.6	19.9	0.63	-0.312	.228	223.8	21.6	2.50	+0.944
BGI-3	.204	209.1	29.1	0.95	-0.497	.214	289.0	30.1	2.84	+0.376
BGI-4	.192	304.2	39.7	1.58	-0.433	.198	344.8	40.2	3.15	+0.024
BGI-5	.173	422.2	58.5	1.91	-0.814	.180	478.7	59.4	3.82	-0.239
BGI-6	.167	534.8	78.5	2.24	-0.857	.173	592.0	79.4	4.16	-0.313

Table 10. Summary of triaxial test data for Garner specimens compacted to modified Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content, %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[a]
BGA-1	10	145.9	98.9	5.88	78.9	.211	.193
BGA-2	20	147.2	99.7	5.77	81.4	.201	.189
BGA-3	30	147.5	99.9	5.46	78.0	.198	.169
BGA-4	40	146.8	99.5	5.61	78.1	.203	.169
BGA-5	60	147.6	100.0	5.70	81.9	.197	.154
BGA-6	80	147.6	100.0	5.78	82.9	.197	.147
Average		147.1	99.7	5.70	80.2		

[a]A. C.: after consolidation.

Table 10. (Continued)

Specimen Number	Minimum Volume Condition: Void Ratio	$\overline{\sigma}_1$ psi	$\overline{\sigma}_3$ psi	Strain, %	Volume Change, %	Maximum Effective Stress Ratio: Void Ratio	$\overline{\sigma}_1$ psi	$\overline{\sigma}_3$ psi	Strain, %	Volume Change, %
BGA-1	.191	85.9	9.3	0.61	-0.226	.208	78.9	12.1	2.15	+1.195
BGA-2	.185	160.8	18.7	0.93	-0.286	.192	245.1	20.0	2.17	+0.302
BGA-3	.165	218.4	28.8	0.93	-0.338	.170	308.9	29.5	2.17	+0.147
BGA-4	.163	282.7	38.2	1.25	-0.544	.168	374.5	38.9	2.49	-0.112
BGA-5	.145	405.0	56.9	1.56	-0.794	.152	504.8	58.2	3.11	-0.202
BGA-6	.133	566.0	76.4	1.87	-1.168	.136	637.2	77.5	2.80	-0.904

Table 11[a]. Summary of triaxial test data for Bedford specimens compacted to standard Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content, %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[b]
SB-1	5	127.1	95.2	10.23	80.9	.345	.327
SB-2	20	126.4	94.7	10.77	83.1	.354	.311
SB-3	30	126.1	94.4	10.14	78.4	.353	.305
SB-4	40	128.4	96.2	9.85	81.2	.331	.278
SB-5	40	126.9	95.1	9.68	76.4	.346	.286
SB-6	60	127.9	95.8	9.35	76.4	.334	.268
SB-7	80	127.5	95.5	10.45	83.4	.342	.263
Average		127.2	95.5	10.06	79.9		

[a]Data from source: (18).

[b]A. C.: after consolidation.

Table 11[a]. (Continued)

Specimen Number	Minimum Volume Condition: Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume Change, %	Maximum Effective Stress Ratio: Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume Change, %
SB-1	.325	47.8	4.5	1.01	-0.144	.327	63.0	5.2	2.02	+0.016
SB-2	.304	131.7	18.3	2.51	-0.519	.308	154.0	20.0	5.46	-0.174
SB-3	.295	201.3	28.2	3.08	-0.745	.296	213.1	28.7	4.65	-0.668
SB-4	.265	238.7	36.1	4.65	-1.031	.265	238.7	36.1	4.65	-1.031
SB-5	.262	271.3	39.2	6.50	-1.878	.263	271.3	39.2	6.50	-1.878
SB-6	.253	374.4	56.9	4.34	-1.175	.253	379.3	56.9	5.07	-1.167
SB-7	.240	454.2	70.7	5.98	-1.820	.240	459.0	71.0	6.61	-1.795

Table 12[a]. Summary of triaxial test data for Gilmore specimens compacted to standard Proctor density

Specimen Number	Lateral Pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[b]
SGI-1	10	132.2	94.2	6.66	60.9	.303	.272
SGI-2	20	133.5	95.2	6.28	59.6	.291	.253
SGI-3	20	132.6	94.5	7.37	67.8	.299	.264
SGI-4	30	132.6	94.5	6.75	61.7	.300	.256
SGI-5	30	136.7	97.4	6.62	70.0	.261	.228
SGI-6	40	131.4	93.7	6.72	59.5	.312	.258
SGI-7	40	136.2	97.0	7.03	89.4	.265	.217
SGI-8	60	134.3	95.7	7.56	73.7	.283	.213
SGI-9	60	132.7	94.6	6.96	84.6	.298	.229
SGI-10	80	135.3	96.4	7.33	69.6	.274	.193
SGI-11	80	135.5	96.6	6.67	67.9	.271	.219
Average		133.9	95.4	6.90	69.5		

[a]Data from source: (18).

[b]A. C.: after consolidation.

Table 12[a]. (Continued)

Specimen Number	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume Change, %	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %
	Minimum Volume Condition					Maximum Effective Stress Ratio				
SGI-1	.267	64.4	9.6	0.88	-0.43	.282	117.0	10.1	3.63	+0.80
SGI-2	.247	172.6	19.8	1.77	-0.46	.249	197.4	20.2	2.84	-0.29
SGI-3	.260	166.8	19.8	1.65	-0.37	.262	206.7	21.1	3.18	-0.15
SGI-4	.246	193.3	27.7	1.93	-0.74	.254	244.7	28.4	4.63	-0.11
SGI-5	.222	239.4	29.9	1.72	-0.48	.224	296.0	31.1	3.45	-0.30
SGI-6	.247	272.3	38.1	2.66	-0.89	.257	313.2	38.6	5.44	-0.05
SGI-7	.207	314.9	39.1	3.40	-0.84	.209	336.9	40.4	4.82	-0.70
SGI-8	.195	375.8	55.6	3.99	-1.53	.203	419.8	57.7	6.80	-0.87
SGI-9	.216	449.3	60.2	5.53	-1.01	.216	449.3	60.2	5.53	-1.01
SGI-10	.179	455.2	77.0	2.43	-1.23	.186	543.8	77.7	4.99	-0.66
SGI-11	.206	508.8	78.4	3.37	-1.06	.208	540.8	79.4	5.12	-0.92

Table 13[a]. Summary of triaxial test data for Garner specimens compacted to standard Proctor density

Specimen Number	Lateral pressure, psi	Density pcf	Per Cent Modified Proctor	Moisture Content %	Initial Saturation, %	Void Ratio Initial	Void Ratio A. C.[b]
SGA-1	10	144.3	97.8	6.48	81.9	.224	.204
SGA-2	10	145.5	98.6	6.51	86.1	.214	.192
SGA-3	20	144.8	98.1	6.49	83.5	.220	.192
SGA-4	20	145.6	98.7	7.36	97.8	.213	.186
SGA-5	30	145.6	98.6	6.83	90.7	.213	.176
SGA-6	30	145.2	98.4	6.58	85.8	.217	.184
SGA-7	40	145.9	98.8	6.85	90.6	.211	.172
SGA-8	40	145.5	98.6	7.16	96.0	.214	.169
SGA-9	60	147.0	99.6	6.42	89.9	.202	.155
SGA-10	80	144.1	97.6	6.99	87.5	.226	.172
Average		145.4	98.5	6.77	89.0		

[a]Data from source: (18).

[b]A. C.: after consolidation.

Table 13[a]. (Continued)

Specimen Number		Minimum Volume Condition					Maximum Effective Stress Ratio			
	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %	Void Ratio	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume Change, %
SGA-1	.201	87.0	9.8	1.30	-0.220	.206	135.6	11.3	2.73	+0.197
SGA-2	.190	80.5	9.8	0.94	-0.136	.197	156.2	12.5	2.63	+0.415
SGA-3	.189	136.6	19.3	1.18	-0.262	.197	219.8	21.4	3.36	+0.406
SGA-4	.181	199.2	19.4	1.92	-0.416	.183	227.6	20.6	2.81	-0.220
SGA-5	.169	272.7	28.9	2.02	-0.655	.173	300.6	29.6	2.77	-0.566
SGA-6	.177	237.9	29.2	1.52	-0.636	.178	315.3	31.6	3.10	-0.306
SGA-7	.165	334.7	39.2	2.09	-0.632	.168	396.5	41.4	3.74	-0.349
SGA-8	.163	371.4	39.2	2.29	-0.586	.164	400.4	40.4	3.13	-0.442
SGA-9	.144	477.7	57.9	2.57	-0.912	.145	516.2	59.4	3.48	-0.854
SGA-10	.159	540.0	76.9	2.59	-1.075	.161	621.8	80.2	4.36	-0.938