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IMPROVEMENT OF GRANULAR BASE COURSE MATERIALS WITH PORTLAND CEMENT

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INTRODUCTION

A highway base course may be defined as a layer of granular material which lies immediately below the wearing surface of a pavement and must possess high resistance to deformation in order to withstand pressures imposed by traffic.

A material commonly used for base course construction is crushed limestone. Sources of limestone, acceptable for highway bases in the state of Iowa, occur almost entirely in the Pennsylvanian, Mississippian and Devonian strata. Performance records of the latter two have been quite good, while material from the Pennsylvanian stratum has failed on numerous occasions.

The study reported herein is one segment of an extensive research program on compacted crushed limestone used for flexible highway base courses. The primary goals of the total study are:

1. Determination of a suitable and realistic laboratory method of compaction.
2. Effect of gradation, and mineralogy of the fines, on shearing strength.
3. Possible improvement of the shear strength with organic and inorganic chemical stabilization additives.

Although the study reported herein deals primarily with the third goal, information gathered from work on the first two was required for this investigation.

The primary goal of this study was the evaluation of various factors of stability of three crushed limestones when treated with small amounts of type I Portland cement. Investigation of the untreated materials has indicated that shear strength alone is not the controlling factor for stability of crushed stone bases. Thus the following observations were made in addition to shear strength parameters, to more adequately ascertain the stability of the cement treated materials:

1. Volume change during consolidation and shear testing.
2. Pore pressure during shear.

The consolidated-undrained triaxial shear test was used for determination of the above factors.

REVIEW OF LITERATURE

Strength of a flexible pavement is derived from distribution of a load over the subgrade through the subbase, base, and surface courses rather than by the load carrying capacity of the pavement as a whole. The base course is often a layer of granular material, immediately below the surface course, whose function is to distribute intense surface loads over a large area of the subgrade.

The primary requirement for an aggregate to be used as base course material is stability, which may be defined as the ability to transfer wheel loads to the underlying layers without permanent deformation (20). The stability of a granular material is dependent upon particle size distribution, particle shape, relative density, internal friction and cohesion (27).

In order to determine the effect of Portland cement on the three materials under investigation it is necessary to understand the sources of variation of stability in granular materials.

Tests have shown that the chemical composition of a material has little to do with the shear strength of an aggregate (16). Instead, the shape and texture of the particles, to an equal degree, are the primary controlling factors affecting the shearing properties of a given aggregate. A variation in any one or both of these properties has a marked effect on the strength of the material (16).

The amount of material passing the No. 200 sieve and the plasticity of the fines have been shown to have an effect on the shear strength of granular materials (17). The result of variations in the amount of material passing the No. 200 was investigated by using 3/4 inch maximum size crushed stone and varying the amount of fines to 1, 4.5, 9, 13, and 20%. With respect to load carrying capacities, it was found that 9% was optimum. Higher or lower values tended to decrease strength. The optimum value was found to decrease as the maximum size of the aggregate increased.

Plastic fines act as a lubricating agent and generally result in larger amounts of strain for given stress conditions (17). There is a rapid increase in the strain required to develop given stress conditions as the plasticity index is increased (17). Cement contents of the order of two to five percent by weight usually reduce the plasticity of granular soils, having indices of the order of 10 to 15, to values of the order of five or less (11).

A granular material is cohesionless but exhibits an apparent cohesion which has been attributed to particle interlocking (13). Volume expansion is necessary in a granular material to allow the interlocking particles to slide up over each other and allow deformation to occur. As sliding begins, the shear stress and rate of volume expansion reach maximum values (18).

The effect of particle interlocking is considered to be of considerable importance in the frictional properties of an aggregate (13). Particle interlocking is achieved through increased density, increase of gravel size content and angular particle shape. The effect of particle interlock is especially significant at low lateral pressures, but is less pronounced as the lateral pressure increases. This condition may produce an envelope of limiting shear resistance which is not a straight line but concave downward, particularly at low stress values (13).

Addition of cement to a soil increases the resistance to frost action, as well as increasing the strength characteristics. Classification of a material as soil-cement indicates that cement content is of sufficient amount to resist frost action as determined by the ASTM Standard D560-57 Freeze-Thaw test (2). The terms cement modified or cement treated are used as a prefix for any material having a cement content less than that required for classification as soil-cement.

Investigations into the effects of cement treatments of crushed limestones are almost nonexistent. Work has been conducted on various gravels which have usually been compared with untreated crushed limestone.

The first known application of stabilization of granular material with cement occurred in 1915 when an interprising contractor in Sarasota, Florida, built a section of Oak Street by dredging shell from the bay, mixing it with sand and cement, using a plow and then compacting the surface with a 10-ton steam roller. Speculation is that the contractor resorted to this unorthodox method of construction after a breakdown of concrete mixing equipment (11).

One of the most significant developments in the field of soil-cement was that the moisture-density relationship for soils was also valid for mixtures of soil and cement when compacted immediately after mixing and prior to cement hydration. It was found that optimum moisture content as determined by moisture-density test not only produced the highest density for a particular compactive effort but also provided sufficient water for cement hydration and maximum strength (11).

Structural properties of soil-cement mixtures are dependent on several factors including the following (10):

1. Physical and chemical properties of the soil.
2. Cement content of the mixture.
3. Moisture content of the mixture.
4. Density of the compacted mixture.
5. Age of specimens and the method of curing.

Factors 1, 2, and 5 are of primary importance for the investigation reported herein.

In more granular soils, the cementing action approaches that of concrete, except that the cement paste does not fill the voids between the aggregate (11). In sands, the aggregate becomes cemented only at points of contact. The more densely graded the soil, the smaller the voids, the more numerous and greater the contact areas, and the stronger the cementing action. Uniform sand has a minimum amount of contact area and requires more cement than well-graded granular materials. Because well-graded granular soils generally have a low swell potential and low frost susceptibility, it is possible to stabilize them with lesser cement contents than are needed for uniformly graded sands, silts and clays. For any type of soil, the cementing process is given maximum opportunity to develop when the mixture is highly compacted at a moisture content that facilitates both the densification of the mix and the hydration of the cement (11).

Investigation into the shear strength of soil-cement mixtures under triaxial loading has been reported by Balmer (4). Results of triaxial shear from laboratory tests on cylinders molded from two granular and two fine-grain soils were presented. The range of lateral pressures used for this study was 0 to 60 psi. This was felt to be a realistic range for road or runway base materials and is probably within the range of stresses developed under traffic. Balmer (4) noted with granular materials that the angles of internal friction were relatively constant and were unrelated to the percent of cement used. Cohesive strength of these materials increased rapidly as the cement content was increased; i.e., higher increases in cohesion with granular than with fine grained soils having the same cement contents. Air-dried specimens showed marked increases in ϕ and c. Balmer (4) noted that as a specimen dried, water films surrounding the particles became very thin and exerted high surface tension forces.

Cement content or age had little influence on Poisson's ratio for any of the cement-treated soils in Balmer's study (4). For granular soils the average value for moist-cured specimens varied between 0.10 and 0.20.

Field tests have been conducted on granular soil-cement and cement-modified mixtures for highway base courses subjected to freezing and thawing (1). The tests showed that the load-carrying capacities of the standard soil-cement mixtures were not adversely affected by exposure for five years to freezing and thawing conditions existing in the Skokie, Illinois area. In contrast, the load-carrying capacities of the cement-modified materials containing the lower cement contents were reasonably high after exposure for one winter, but were reduced during the 5-year test period. The capacity of the cement-modified materials to support loads, remained significantly greater than those of the untreated soils of the same thickness, however.

Granular materials containing low cement contents may be used to a distinct advantage over substandard granular materials in all climates, though their greatest advantage is in climates where freezing-and-thawing is not severe (1). However, additional field tests are needed to develop specifications for the proper and effective use of these materials both in "frost" and "nonfrost" regions.

MATERIALS

Three crushed stones were used in this project. Each was selected in cooperation with the Iowa State Highway Commission 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 obtained from near Bedford, Taylor County, Iowa. Hereafter referred to as the Bedford sample. The system outcrops in nearly half of the state. Formations in this system are generally quite soft and contain relatively high amounts of clay.
2. A hard limestone obtained from near Gilmore City, Humbolt County Iowa. Hereafter referred to as the Gilmore sample. This material is from the Mississippian System which outcrops in a rather discontinuous and patchy band across the center of the state. Formations are quite variable but contain ledges of concrete quality rock.
3. A hard dolomite obtained from near Garner, Hancock County, Iowa. Hereafter referred to as the Garner sample. From the Devonian System, this material is very uniform and has shown remarkable similarity through several counties.

Having met Iowa State Highway Commission Specifications, the three crushed limestones were tested in the same condition that they were received from the quarry stockpile, i.e., physical and chemical properties were in no way altered upon receipt.

Chemical and mineralogical properties of the three stones as determined by X-ray identification, measurement of pH, cation exchange capacity (C E.C.), and hydrochloric acid soluble and non-soluble minerals are shown in Tables 1, 2, and 3. Table 4 presents the engineering properties of each of the three materials.

The cement used for this investigation was a Type I Portland cement obtained locally.

Prior to the investigation of the shear strength of the Portland cement treated crushed limestones, investigations were conducted on the freeze-thaw durability of the treated material (14). The ASTM brushing loss test showed that the required cement content for classification as soil-cement was 5,3 and 3% by weight for the Bedford, Garner, and Gilmore samples, respectively. Throughout the remainder of this investigation, the 3% Garner and Gilmore treatments are the only series that can be classified as true soil-cement. The remaining treatments are classified as cement-modified material.

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

Stone des.	Calcite	Dolomite	Quartz	Feldspars	Calcite/dolomite ratio[a]
Bedford	Pred.	Small amount	Trace	Not ident.	25
Garner	Pred.	Second pred.	Trace	Not ident.	1.16
Gilmore	Pred.	None	Trace	Not ident.	

[a]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

* Representative sample was ground to pass No. 100 sieve.

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

METHOD OF INVESTIGATION

Program of Study

The investigative program was established to determine the effect of small amounts of cement on the overall stability of the three crushed stone materials. The cement contents to be used were set at 1% and 3% by dry weight. Previous investigations in this range of cement contents for use with crushed limestone are quite limited. Field tests have shown that cement modified crushed limestone performs satisfactorily and is of considerable benefit in improving frictional properties (1).

The selected method of testing was the consolidated-undrained triaxial shear test. For each of the three materials, a series of six specimens were tested with 1% and 3% cement following 7 and 28 day curing. Specimens in each series were tested at lateral pressures of 10, 20, 30, 40, 60, and 80 psi. This range of lateral pressures appears to be representative of the conditions occurring in most base courses.

Testing Procedure

Moisture-density relationships obtained from standard Proctor density tests on the cement treated material were not used in this investigation. Initially several specimens were compacted at optimum moisture content as determined by the standard Proctor density test but it was not possible to achieve standard Proctor density. Moisture-density tests were then conducted using the vibratory compactor which resulted in a slightly different optimum moisture content while achieving standard Proctor density.

These moisture-density relationships were then used for the preparation of test specimens.

Table 5 shows the moisture-density relationships for the three materials at the two cement contents for vibratory compaction and the standard Proctor density of the untreated material. It is readily apparent that there is little variation in density due to the method of compaction or the addition of cement.

Table 5. Moisture-density relationships for three materials at two cement contents

	Bedford		Garner		Gilmore	
	Opt. M.C.	D.D.	Opt.	D.D.	Opt. M.C.	D.D.
Standard Proctor untreated	10.9	127.4	7.6	140.5	9.4	130.8
Vibratory						
1% Cem.	10.2	127.6	6.6	138.4	9.8	131.0
3% Cem.	9.7	128.3	5.7	135.1	9.0	133.5

An adequate quantity of crushed stone to produce one specimen, plus 300 gm. for eventual moisture content determination, was air dried, then placed in a sealed container until time of molding. Cement for each specimen was individually weighed and placed in sealed containers.

Prior to molding, the crushed stone and cement were dry mixed by hand for uniform distribution of the cement and prevention of particle degradation. The necessary quantity of water was then added and hand mixing was continued. Following mixing, the material was allowed to mellow in a moist atmosphere for ten minutes after which the material was again hand-mixed and a 150 gm moisture sample removed.

Each specimen was compacted by vibration in a four-inch diameter by eight-inch high cylindrical mold attached to a Syntron Electric Vibrator table. The material was placed in the mold in four equal layers and rodded 25 times per layer with a 3/4 inch diameter, rounded tip rod. A constant frequency of 3600 cycles/min. and amplitude of 0.368 mm were used with a surcharge weight of 35 lb for a period of two minutes. Previous work has shown that this method of compaction is capable of achieving standard Proctor density with a minimum amount of degradation and segregation of the specimen (14). The last 150 gm of the mix was used for final moisture determination.

After compaction, height of the specimens was measured while in the mold. They were then extruded, weighed, wrapped in two layers of Saran wrap and aluminum foil, and the ends sealed. The specimens were then cured for the required periods in an atmosphere of about 75°F and near 100% relative humidity. Prior to testing each specimen was again weighed and the height and diameter measured.

The double bay testing machine used in this study was fabricated by the I.S.U. Engineering Shop to specifications established by the Soil Research Laboratory (Figure 1). Rate of deformation of specimen is variable from about 0.0001 to 0.1 inch/min. Axial load capacity is 11,000 pounds per cell, and is determinable by proving rings.

Positive and negative pore water pressures were measured with Karol-Warner Model 53-PP pore pressure units. Change of specimen volume was measured by a device also developed by the Soil Research Lab, and is capable of precisions of near 0.01 cubic inch.

Fig. 1a. Triaxial test cell, pore pressure unit, volume change device

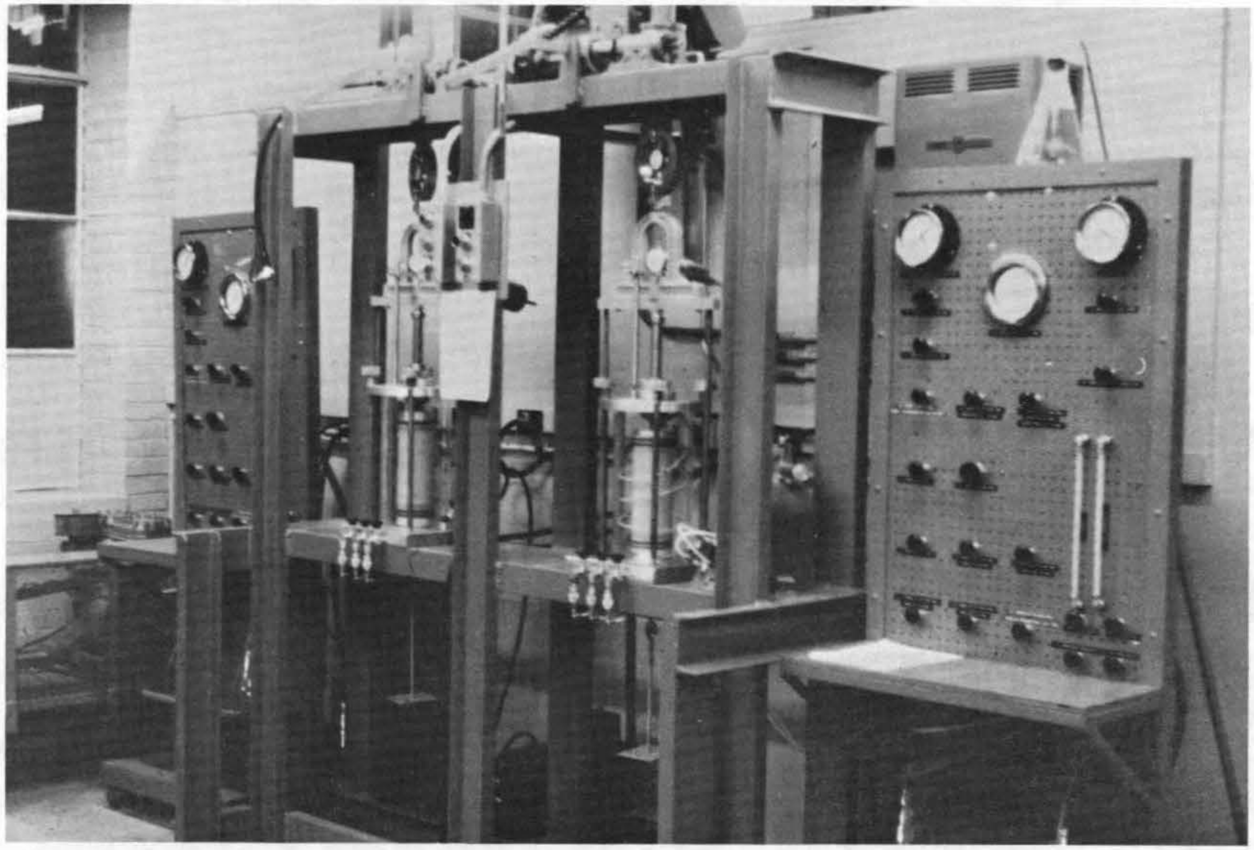

Figure 1. Triaxial shear testing machine

Specimens were sealed in a 0.025 inch thick, seamless, rubber membrane with saturated 1/2 inch thick corrundum porous stones on top and bottom. The triaxial cell was filled with de-aired water to within about 1½ inch of the top, prior to consolidation. A flexible line running from the base of the cell to the bottom of the volume change device allowed flow of water between the two. Initial water level in the volume change device was adjusted by raising or lowering the device until the water level in the cell and the volume apparatus coincided with the cross hair in the eyepiece of the volume change device. As the specimen changed volume, the device was lowered or raised until the water level was again at the initial level. The volume change at this time was equal to the distance the device was moved times the interior cross-sectional area of the tube.

A rate of axial deformation of 0.01 inch/min. was used for all tests, producing a rate of strain of approximately 0.1% per min. Readings of pore pressure, volume change, and axial load were taken at increments of 0.025 inch of axial deflection.

In the early stages of testing, attempts were made to continue the test until a constant specimen volume and/or pore pressure was reached. It was soon noted that total deflection of more than one inch resulted in a ruptured membrane, loss of the specimen and rapid loading of the pore pressure apparatus. Remaining tests were therefore terminated at maximum of one inch deflection.

ANALYSIS OF RESULTS

Failure Criterion

Initial step in the analysis of test results was to establish a criterion for failure. A large number of triaxial investigations have been analyzed on the basis of maximum deviator stresses, $(\sigma_1-\sigma_3)$, as the condition of failure. Holtz (12) reported in 1947 that this criterion of failure was valid where complete drainage can be developed during testing or if pore pressure is not developed within the specimen during the test. When pore pressure exists within a specimen, the concept is no longer applicable. He proposed the maximum effective stress ratio $\frac{\sigma_1-\sigma_3}{\sigma_3}$ or $\frac{\sigma_1}{\sigma_3}$ as the "true" failure criterion when pore pressure exists within a specimen during shear.

Shearing strength of a soil, assuming only frictional resitance, is dependent upon the contact pressure between the soil grains. Presence of pore water pressure alters the contact between grains and thus affects the resistance to shearing.

Loading of a granular soil specimen results in a volume decrease initially, after which expansion begins, resulting in a decrease in pore pressure, and a corresponding increase in effective lateral pressure. The increase in the effective lateral pressure results in a gain of axial strength even though failure may have already begun. Holtz (12) states that because of this type of failure, "the maximum principal stress ratio appears to represent the most critical stress condition of the point of incipient failure under variable effective axial and lateral stresses."

With regard to volume change, Holtz (12) made the following statement:

> 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.

The materials used by Holtz (12) were fine sand and sandy clay. Cement treated granular material used for the investigation reported herein did not follow the method of failure described by Holtz. After attaining the point of minimum specimen volume, the effective stress ratio continued to increase and a maximum value was achieved only after expansion had occurred. As mentioned previously, granular materials are capable of developing large resistances to shear by the phenomena of interlocking. Expansion occurs as the particles begin to slide over each other and as sliding just begins, the shear stress and rate of volume expansion reach a maximum value. This indicates that the difference in shear strength at minimum volume, and at maximum effective stress ratio may be an indication of the amount of interlocking within a granular material.

Analysis of results reported herein will be based on both maximum effective stress ratio and minimum volume change as primary conditions of failure. Results for both methods will be compared with the untreated material and further justification for the minimum volume criteria as a condition of failure will be made.

Shear Strength Criteria

The most common method of expressing the shear strength of a soil is by use of the Coulomb equation:

$$\tau = c + \sigma_n \tan \phi$$

in which τ is the shear strength, c is the cohesion, σ_n is the normal stress on the failure plane, and ϕ is the angle of internal friction. As the above equation indicates, the shearing strength is not a simple attribute of the material but is a function of the normal stress. On the other hand, the envelope of failure described by ϕ and c is a function of the material. The shear strength of a granular material is dependent upon the frictional forces developed at the contact points between the grains. These are a function of the effective normal stress rather than the total stress. The Coulomb equation modified for effective stresses becomes:

$$\tau = c' + (\sigma_n - u) \tan \phi'$$

in which u is the pore pressure, c' and ϕ' are in terms of effective stress. For this investigation, the shear strength of the material was analyzed, in terms of ϕ' and c', by three methods.

The Mohr diagram was the first method used for analyses and was constructed using the effective stresses obtained at the point of maximum effective stress ratio. It was readily evident that the results were not of textbook form, and that the determination of the tangential envelope of failure would be difficult.

To obtain a better means of studying the stress conditions, a second method of analysis was used. This was a modified Mohr-Coulomb diagram in which $\frac{1}{2}(\bar{\sigma}_1-\bar{\sigma}_3)$ was plotted against $\frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$ at every point measured during testing[a]. The advantage of this method is that the stress conditions are represented by a series of points instead of a circle, enabling more accurate positioning of the failure envelope. The slope of the resulting failure envelope is designated as tan α, where α is the slope angle from horizontal, and the ordinate intercept as y. The modified shear parameters can be converted to ϕ' and c' by using the following equations:

$$\sin \phi' = \tan \alpha , \quad c' = \frac{y}{\cos \phi'}$$

Plotting the stress conditions to the point of failure represents a stress history of the material, and shows the method of stress build-up.

The third method used, was the Bureau of Reclamation method of least squares. This is a mathematical process of determining the tangent line in terms of ϕ' and c' and assumes a straight-line envelope of failure in that all results are on a common failure envelope. Variations in the strength of individual specimens tend to alter the strength parameters determined by this method, whereas with the modified Mohr-Coulomb method, these variations are easily noticed and the results are not affected by specimen variation.

The modified Mohr-Coulomb diagram was used for visual analysis and determination of the validity of results. The Bureau of Reclamation method was used for the determination of the

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[a] $\bar{\sigma}_1$ and $\bar{\sigma}_3$ represent the maximum and minimum effective principal stresses, respectively.

shear strength parameters.

Specimen Variation

During analyses of the shear strength of the cement treated crushed stones by means of the modified Mohr-Coulomb diagram it was observed that there were minor discontinuities in the failure envelopes. These discontinuities were initially attributed to small variations in density of the specimens and two methods were used to determine if such was the cause of the irregularities. First, additional specimens were tested under the same conditions of length of cure and lateral pressure as the initial specimens that appeared erratic. Though tested under identical conditions, the specimens again indicated some variations, though it was noticed that the amount of variation tended to decrease at the conditions of higher lateral pressure.

Second, a separate study was conducted on the Bedford crushed stone with 3% cement and 7 day cure. The objective of this study was to determine the effect, if any, of variations in density and the consolidation effect of the lateral pressure.

Effect of variations in density, within the range of standard Proctor density of ± 2 pcf, was observed by testing several identical specimens at the same lateral pressure. A total of ten tests were conducted at a lateral pressure of 10 psi. A plot of major principal effective stress against density indicated no noticeable relationship within the range of density used.

The effect of the consolidating pressure was determined by consolidating specimens at 80 psi, reducing the lateral pressure to 10 psi and then shearing the specimen. A total of five tests were conducted in this manner. Maximum effective stress was unaffected but the shape of the stress-strain curve was altered as shown in Figure 2.

It was felt that the variations in strength were primarily due to uneven distribution of cement within the specimen. It was observed during compaction that the fines did not always remain evenly distributed within a specimen. Since the cement would tend to undergo the same pattern of movement as the fines, an uneven distribution of fines should indicate an uneven distribution of cement.

The fines in the cement treated Bedford crushed stone tended to migrate to the top of each specimen and varying amounts were ejected from the mold. Along with the fines, a very small amount of cement was undoubtedly ejected, resulting in a slight reduction of the cement content within the specimen. The total amount of material ejected from the mold was not uniform (i.e., ranged from none to several grams) but tended to vary with each specimen.

The Gilmore crushed stone had a migration of fines to the base of the mold during compaction, probably resulting in a slightly higher concentration of cement in the base and some deficiency at the top. The Garner crushed stone showed no evidence to indicate movement of fines.

Migration of fines could be a result of the amount of fines and the moisture conditions present in the specimen(s). Garner curshed stone had a low optimum moisture content and therefore had less tendency to eject water and fines from the mold during compaction. Bedford and Gilmore materials had higher optimum moisture contents and therefore had a greater

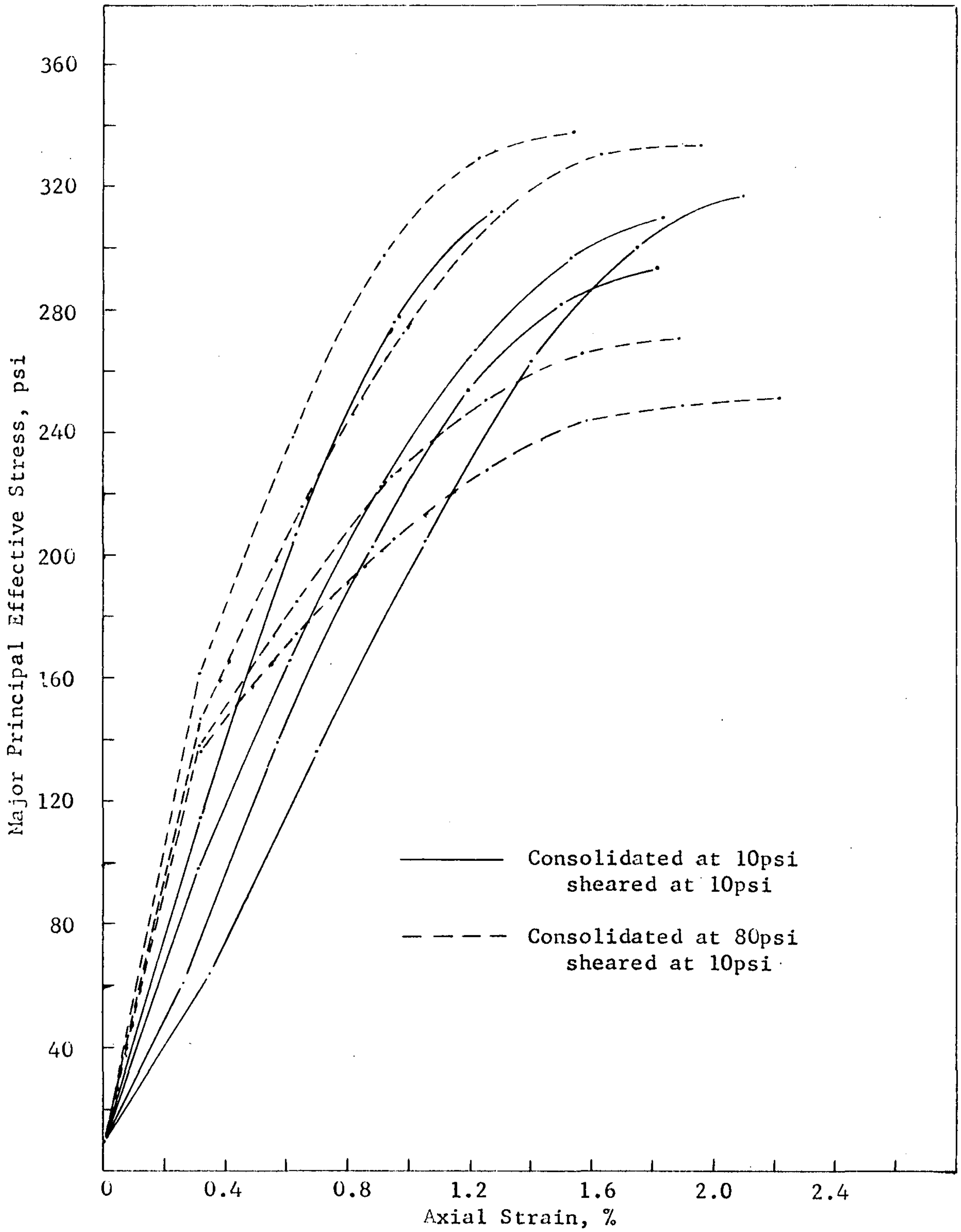

Figure 2. Effect of consolidating pressure on stress-strain characteristics for Bedford, 3% cement treatment, 7-day cure.

tendency for migration of water, fines and cement during compaction.

A possible reason for the migration of fines to the base of the Gilmore specimen may have been the low amount of fines present. Evidently the quantity of fines was not sufficient to fill the voids between the larger aggregates. Vibration during compaction caused movement of fines to the lower portion of the specimen resulting in a smooth uniform appearance at the base while the top of the specimen was rough and somewhat lacking in fines.

The Bedford crushed stone had a larger amount of fines than the Gilmore with 15.6% passing the no. 200 sieve. Evidently this amount of fines was excessive as indicated by ejection of fines from the mold.

Migration of the fines during compaction may not be the important factor, but the fact that the cement additive may follow the same pattern of movement is important. A loss of fines would indicate a reduction in cement content whereas a concentration of fines would indicate an increase of cement, both resulting in potential variations in strength.

The minor change in shear strength due to variation of individual specimens, however, did not account for the discontinuities in the modified Mohr-Coulomb diagram. Figure 3 shows the modified Mohr-Coulomb diagram for the Bedford crushed stone treated with 1% cement and cured for 7 days. The limiting envelope is shown, as well as the stress conditions for equal increments of strain. Specimens sheared under conditions of σ_3 = 10, 20, and 30 psi appear to fall on a common line, while the three remaining specimens fall on another envelope of failure. Thus, the pattern of stress increase appears to be the same for the first three specimens

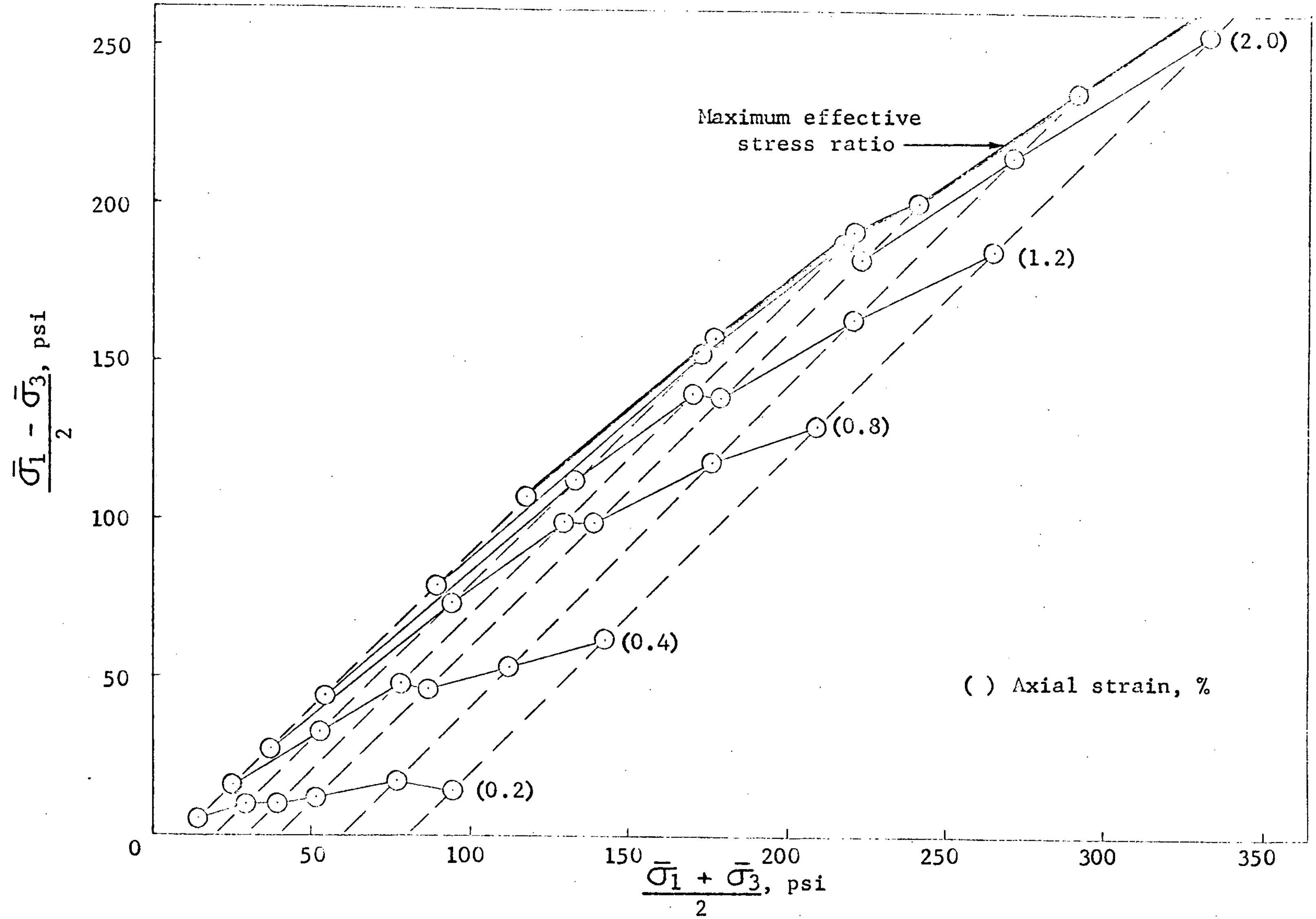

Figure 3. Modified Mohr-Coulomb diagram for Bedford, 1% cement treatment, 7-day cure with stress conditions at equal values of axial strain

while the remaining specimens follow a different pattern.

The above mentioned conditions suggest that the specimens undergo some form of alteration between the conditions of σ_3 = 30 psi and σ_3 = 40 psi. Analysis of the consolidation data indicates what appears to be a preconsolidation confining pressure at about 40 psi, resulting in a definite break in the consolidation curve. It is thus possible that changes resulting from consolidation have an effect on the shear strength of the specimen.

Discontinuities in the modified Mohr-Coulomb diagrams did not appear in the same form for all conditions of material, cement content, and length of cure. In all cases however, the irregularities appeared to be very subtle. Due to the somewhat limited number of specimens tested in each series, it is possible only to suggest the presence of a non-linear envelope of failure. The remaining analysis of results assumes the envelope to be linear, however, and treats these minor irregularities as variations in specimens.

Shear Strength

The modified Mohr-Coulomb diagrams for the cement treated materials are shown in Figures 4 to 15. The envelopes of failure are for conditions of maximum effective stress ratio, and minimum volume for the cement treated material, and for conditions of maximum effective stress ratio only, of the untreated material.

Failure envelopes for the Bedford stone appear to be relatively parallel in all cases, indicating that the angle of internal friction is independent of the amount of cement present, and the length of cure. Additional cement results mainly in a separation of the envelopes of the cement treated stone from that of the untreated material, indicating a change in cohesion.

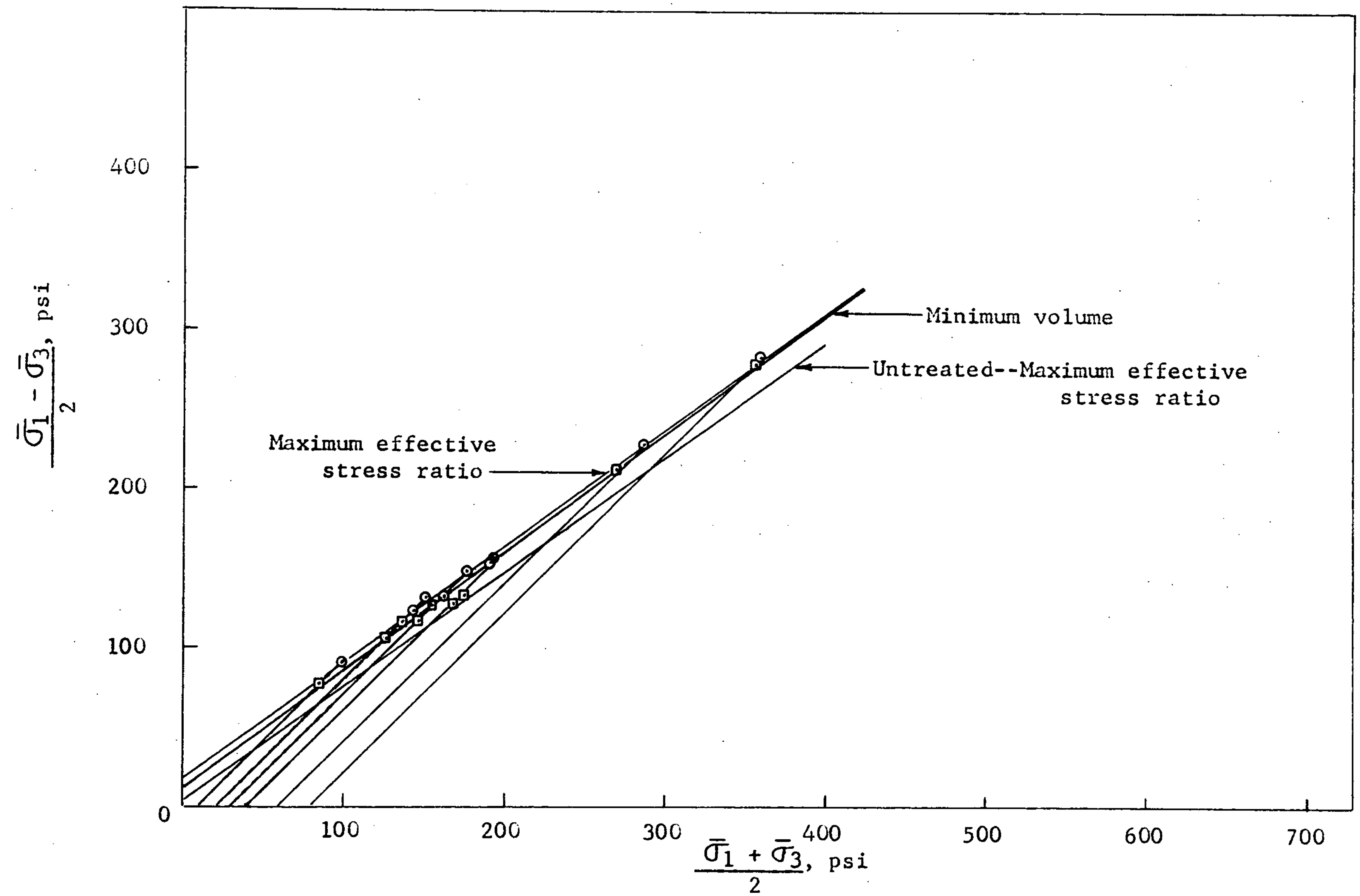

Figure 4. Modified Mohr-Coulomb diagram for Bedford, 1% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

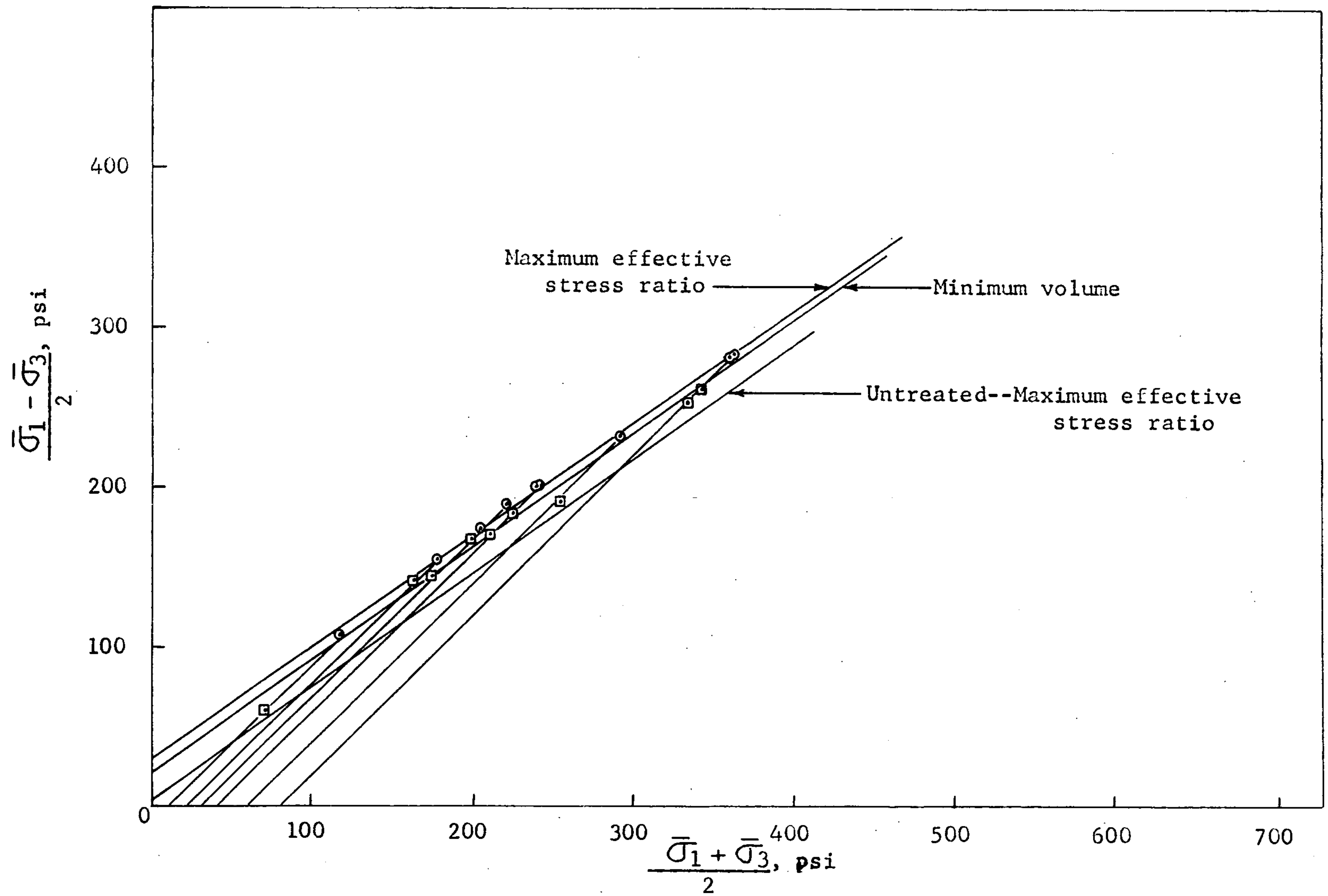

Figure 5. Modified Mohr-Coulomb diagram for Bedford, 1% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

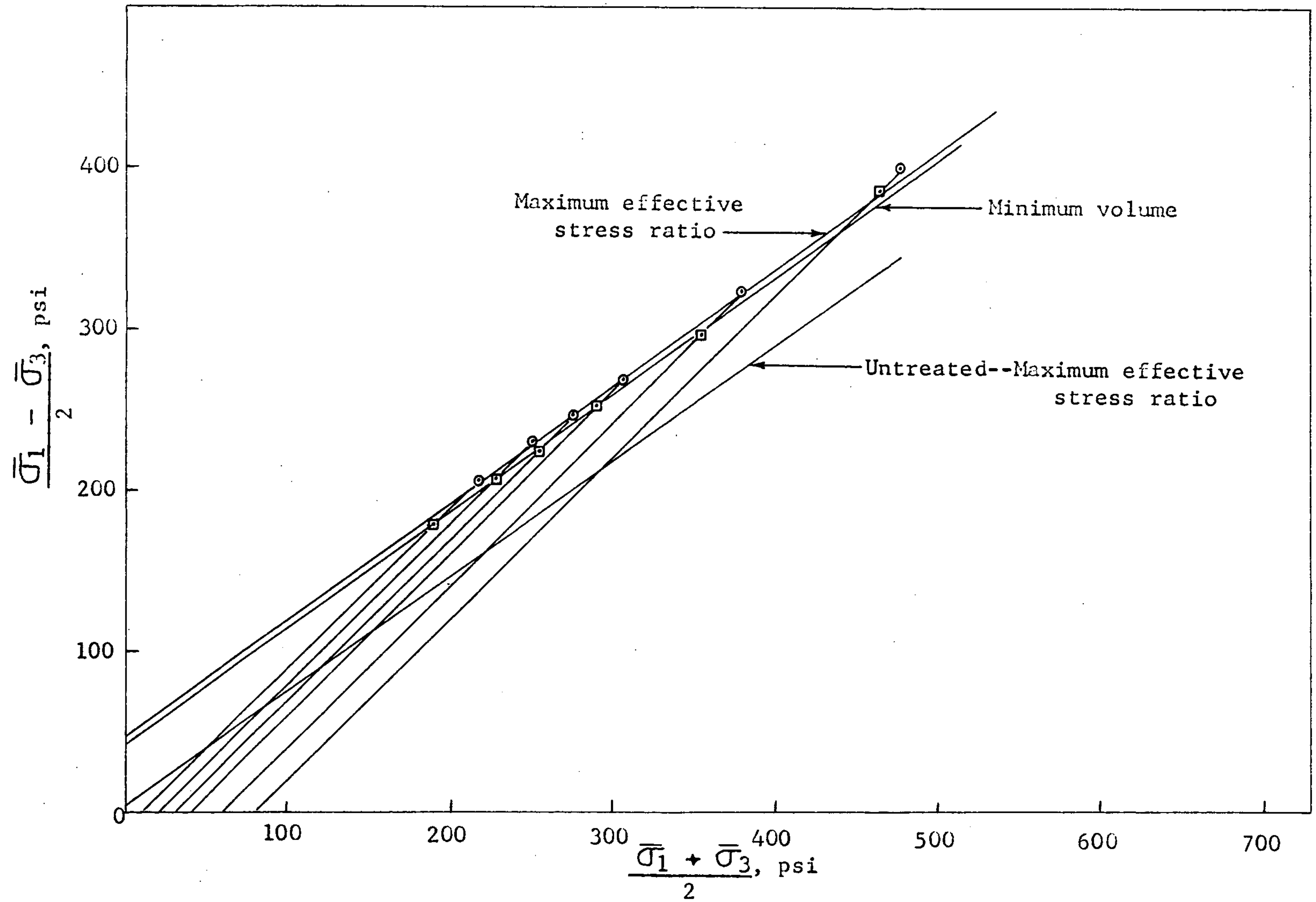

Figure 6. Modified Mohr-Coulomb diagram for Bedford, 3% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

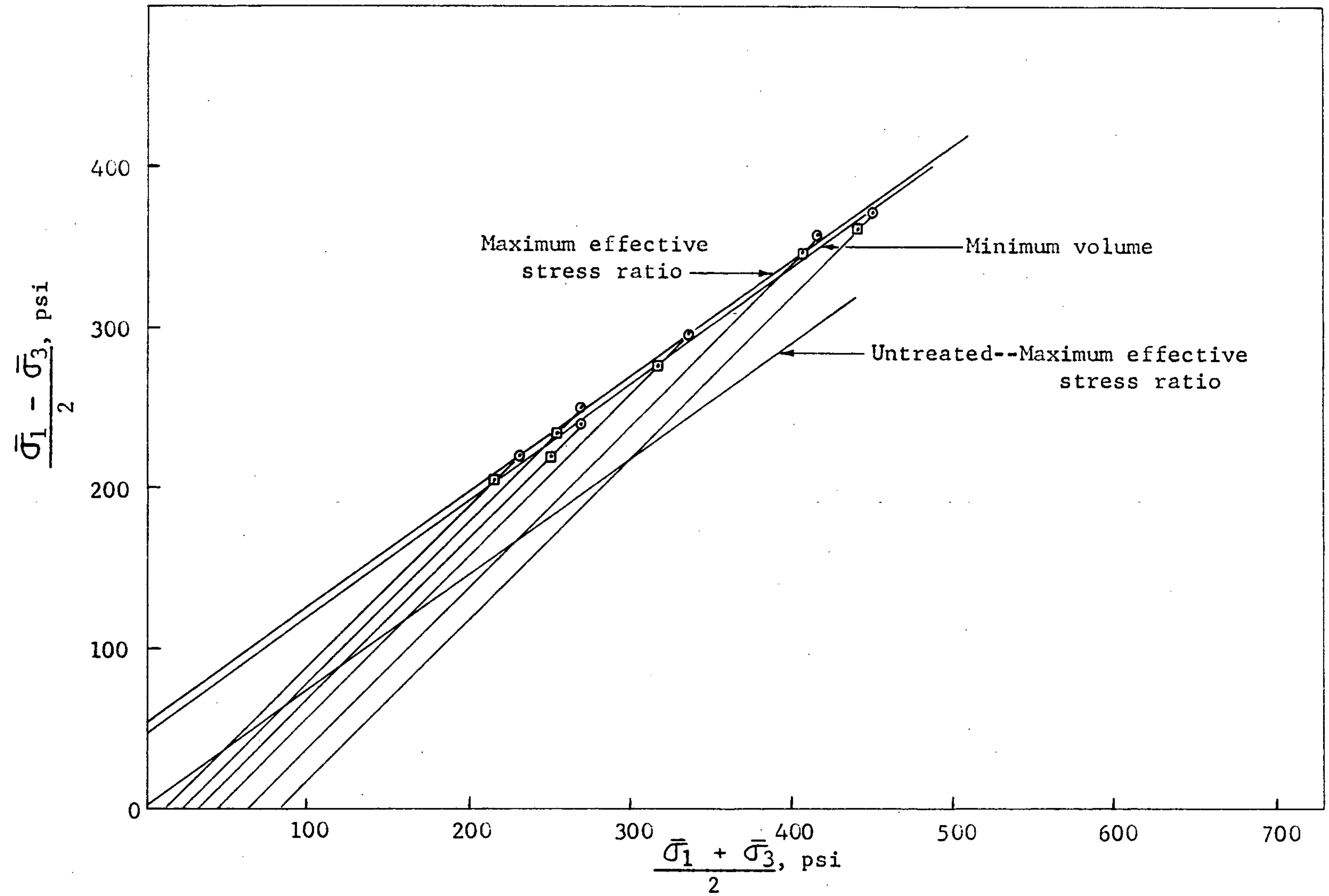

Figure 7. Modified Mohr-Coulomb diagram for Bedford, 3% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

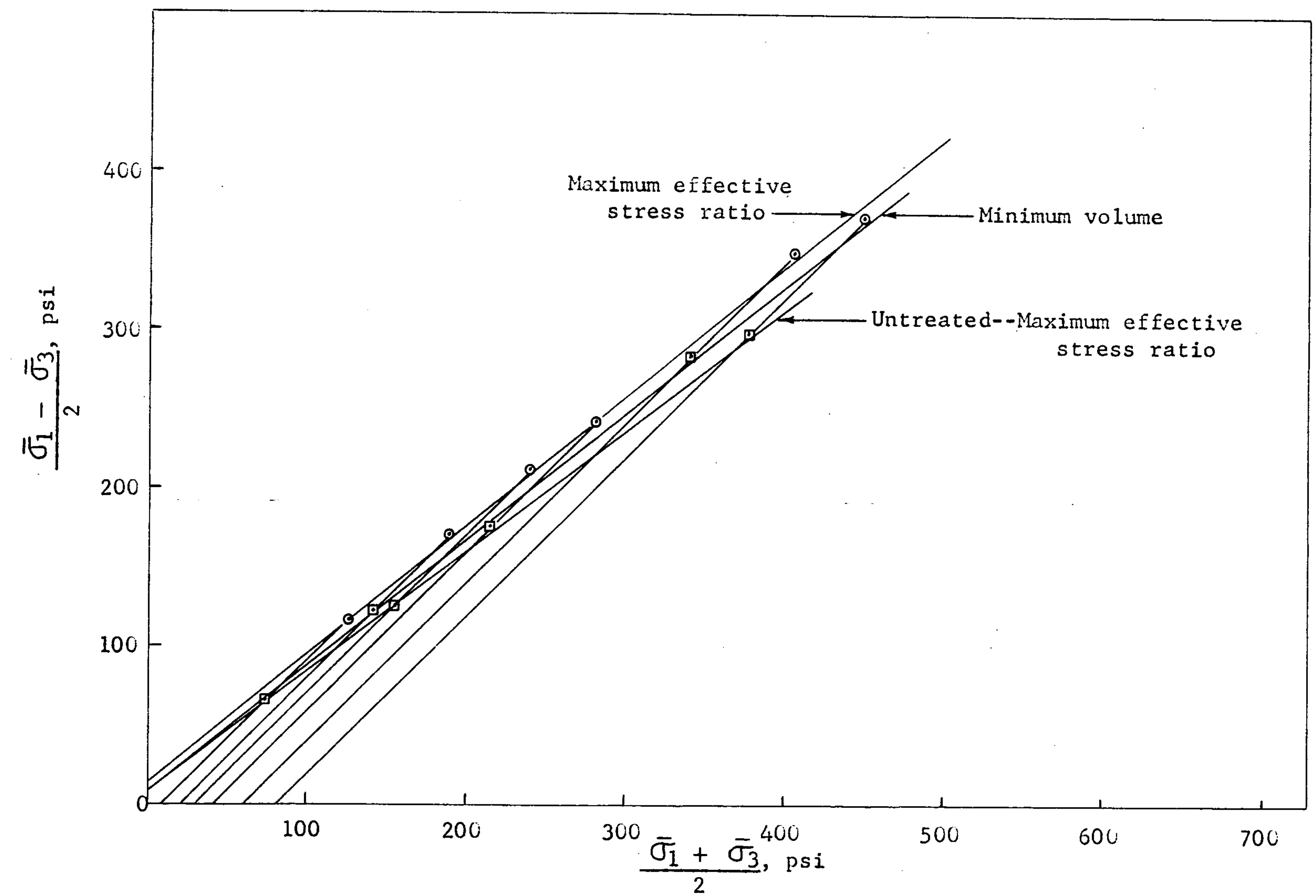

Figure 8. Modified Mohr-Coulomb diagram for Garner, 1% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

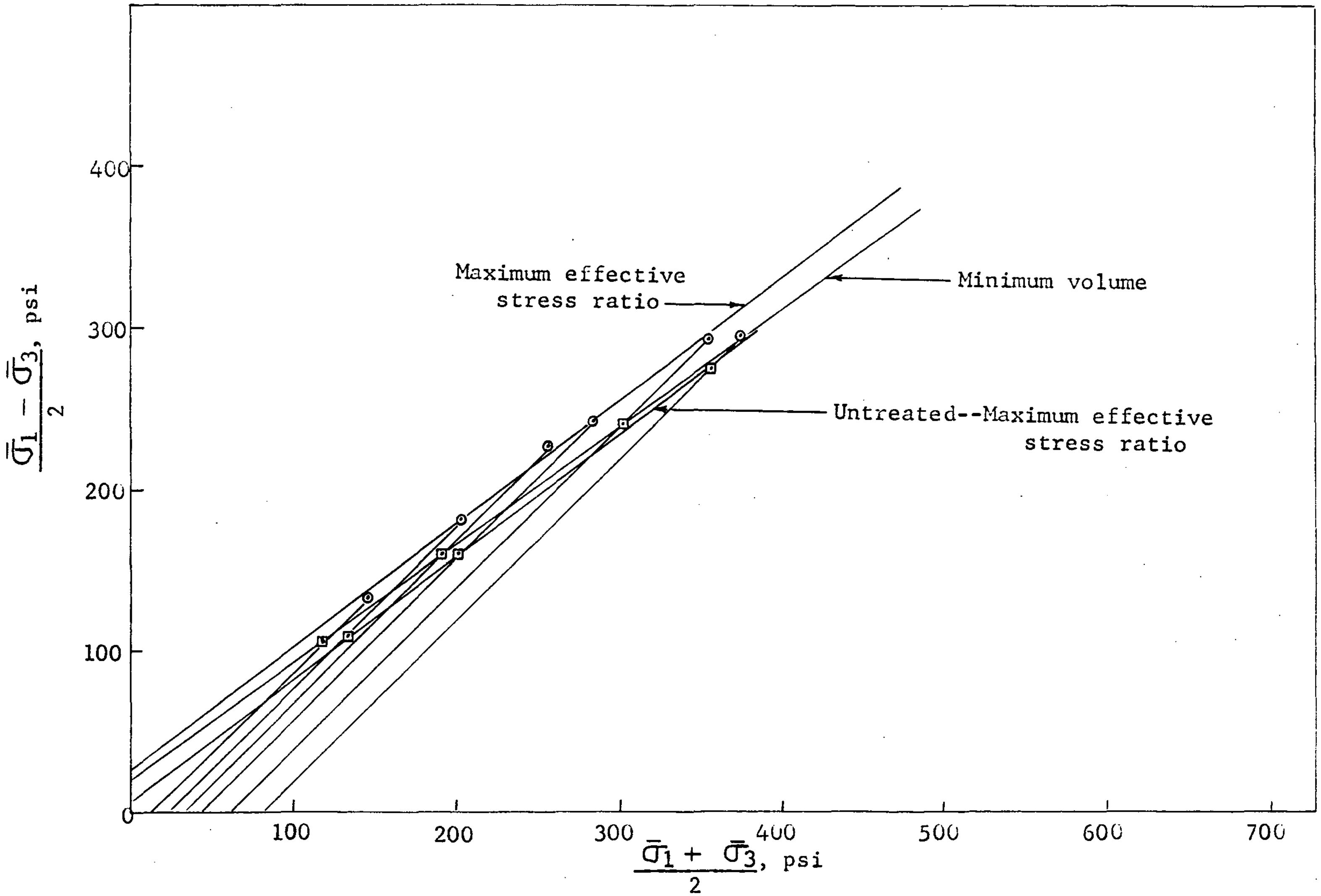

Figure 9. Modified Mohr-Coulomb diagram for Garner, 1% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

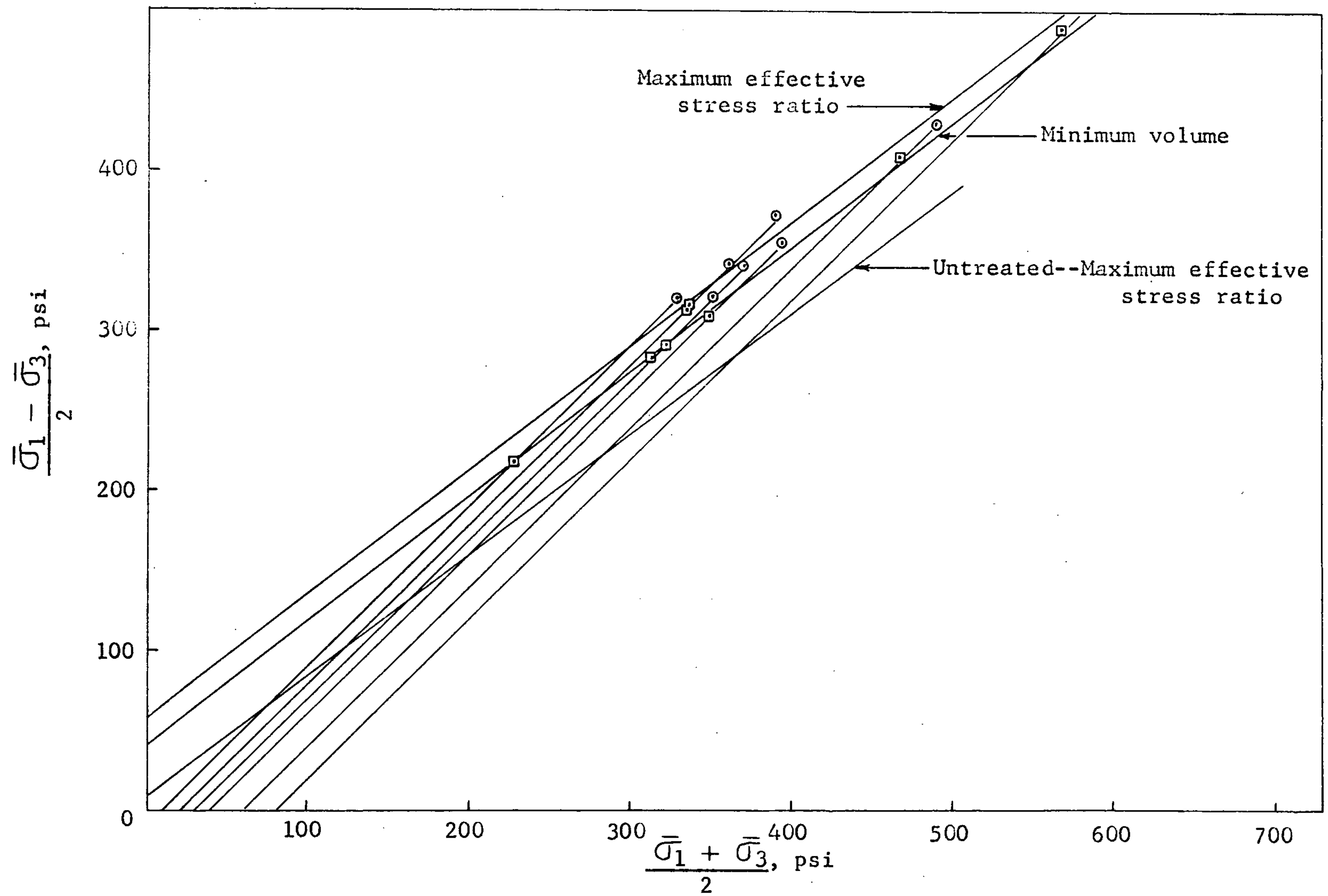

Figure 10. Modified Mohr-Coulomb diagram for Garner, 3% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

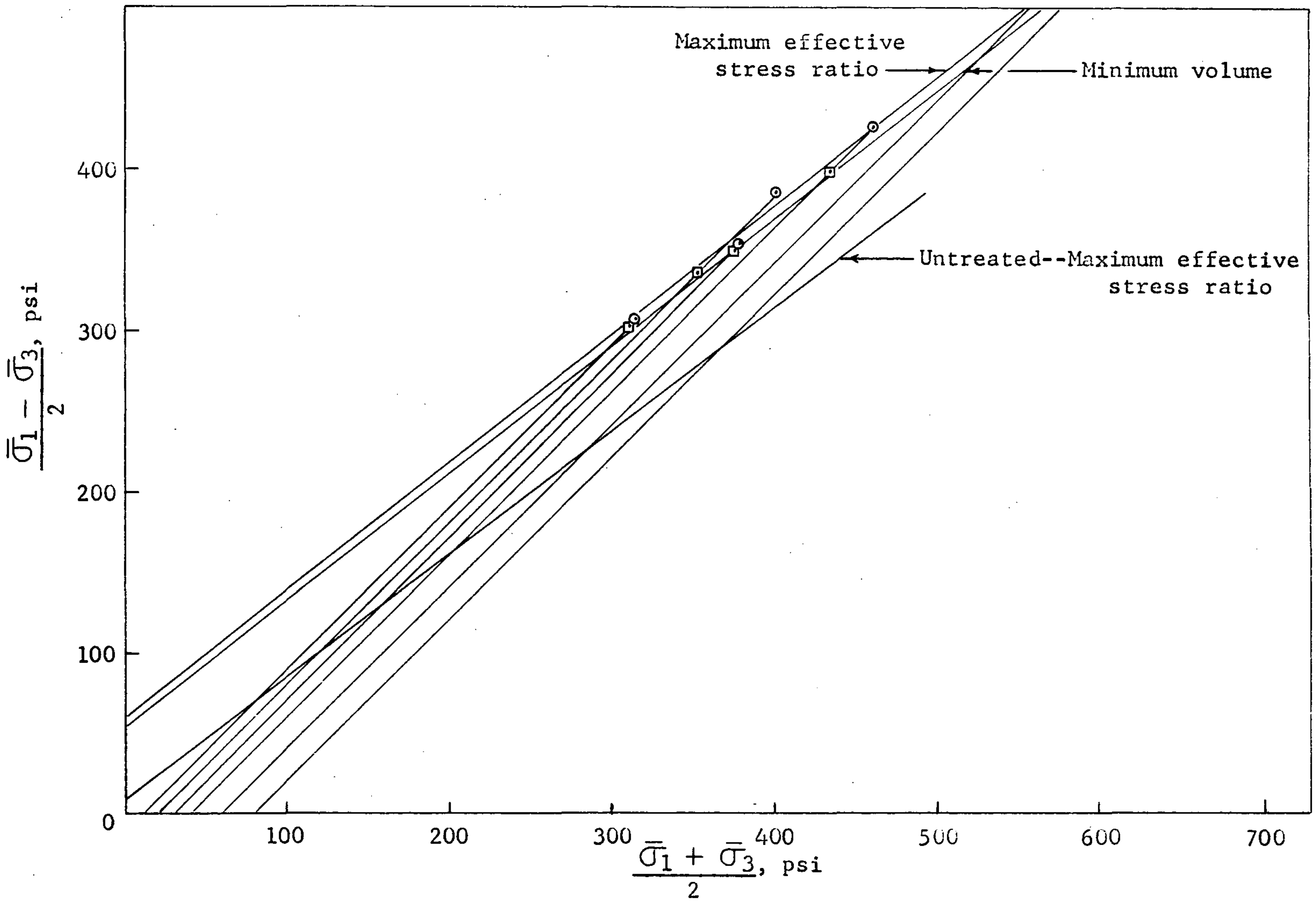

Figure 11. Modified Mohr-Coulomb diagram for Garner, 3% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

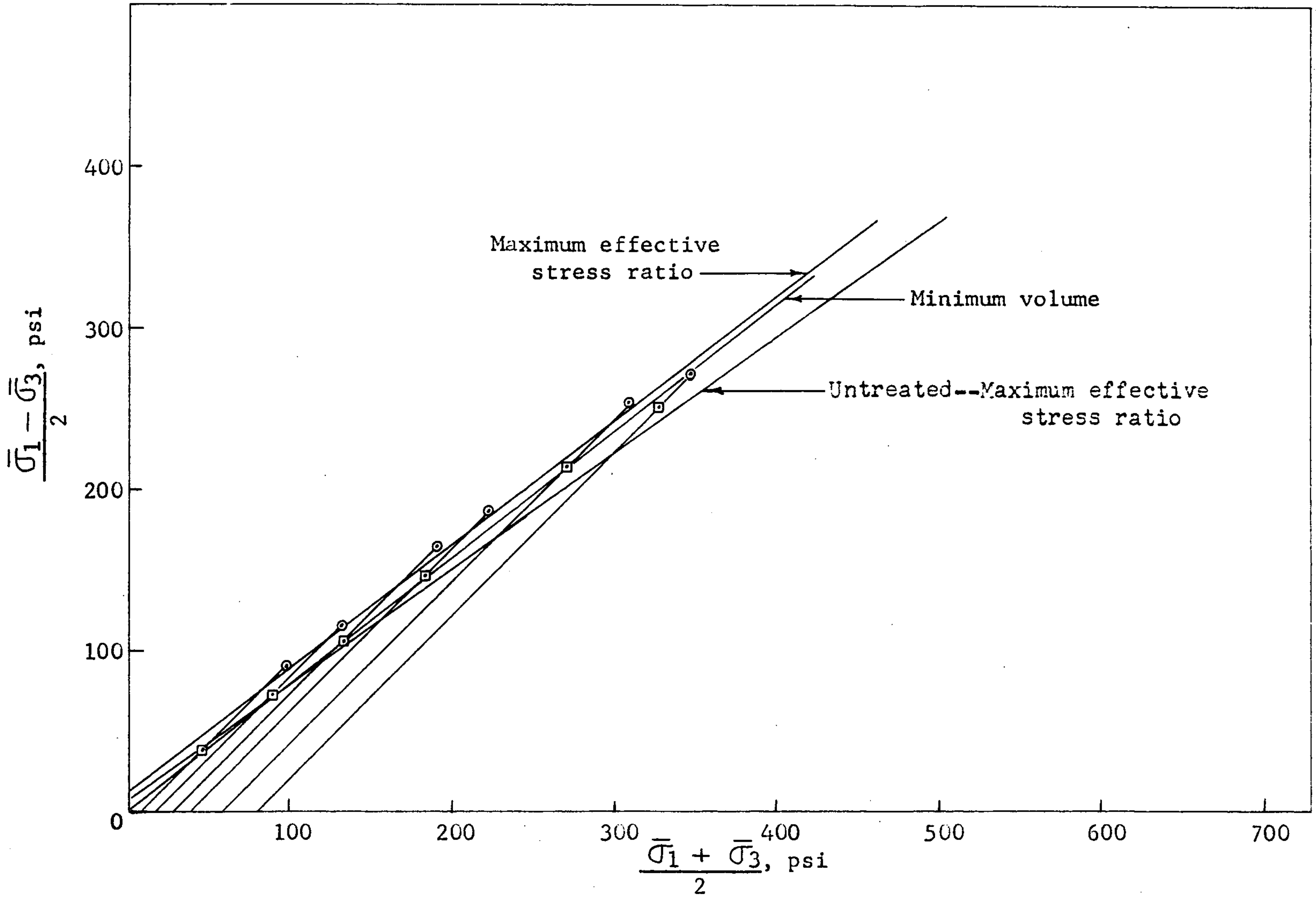

Figure 12. Modified Mohr-Coulomb diagram for Gilmore, 1% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

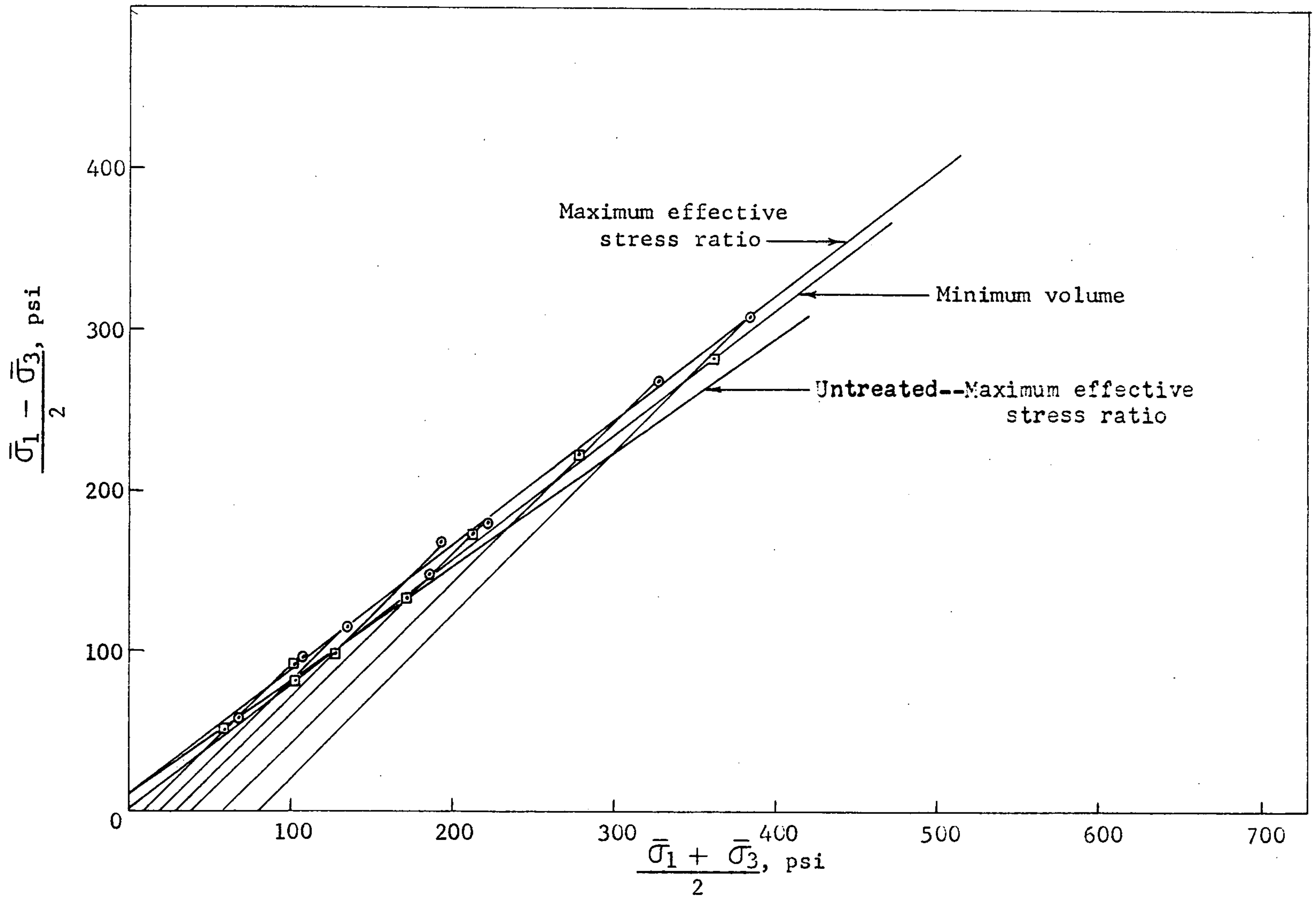

Figure 13. Modified Mohr-Coulomb diagram for Gilmore, 1% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

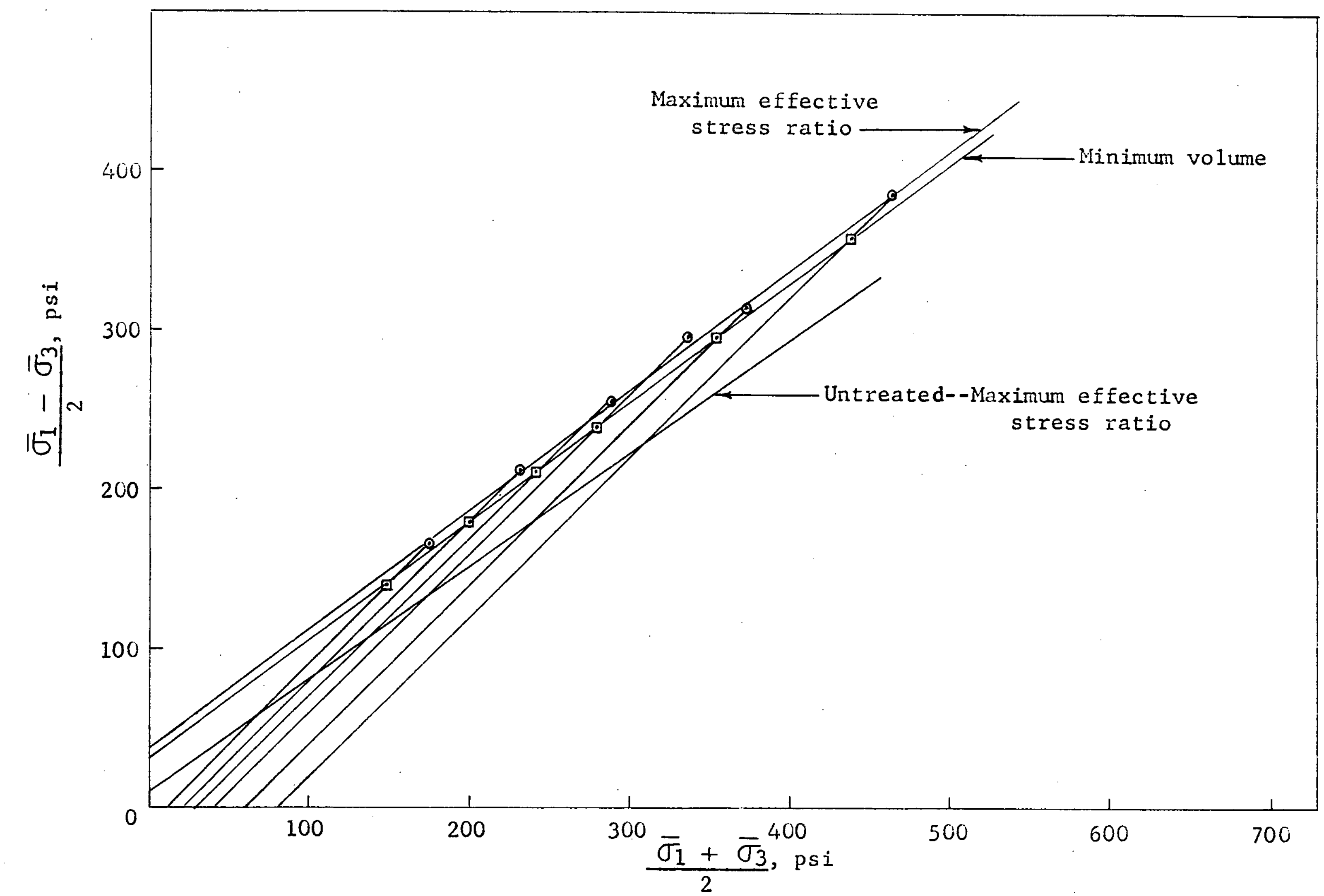

Figure 14. Modified Mohr-Coulomb diagram for Gilmore, 3% cement treatment, 7-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

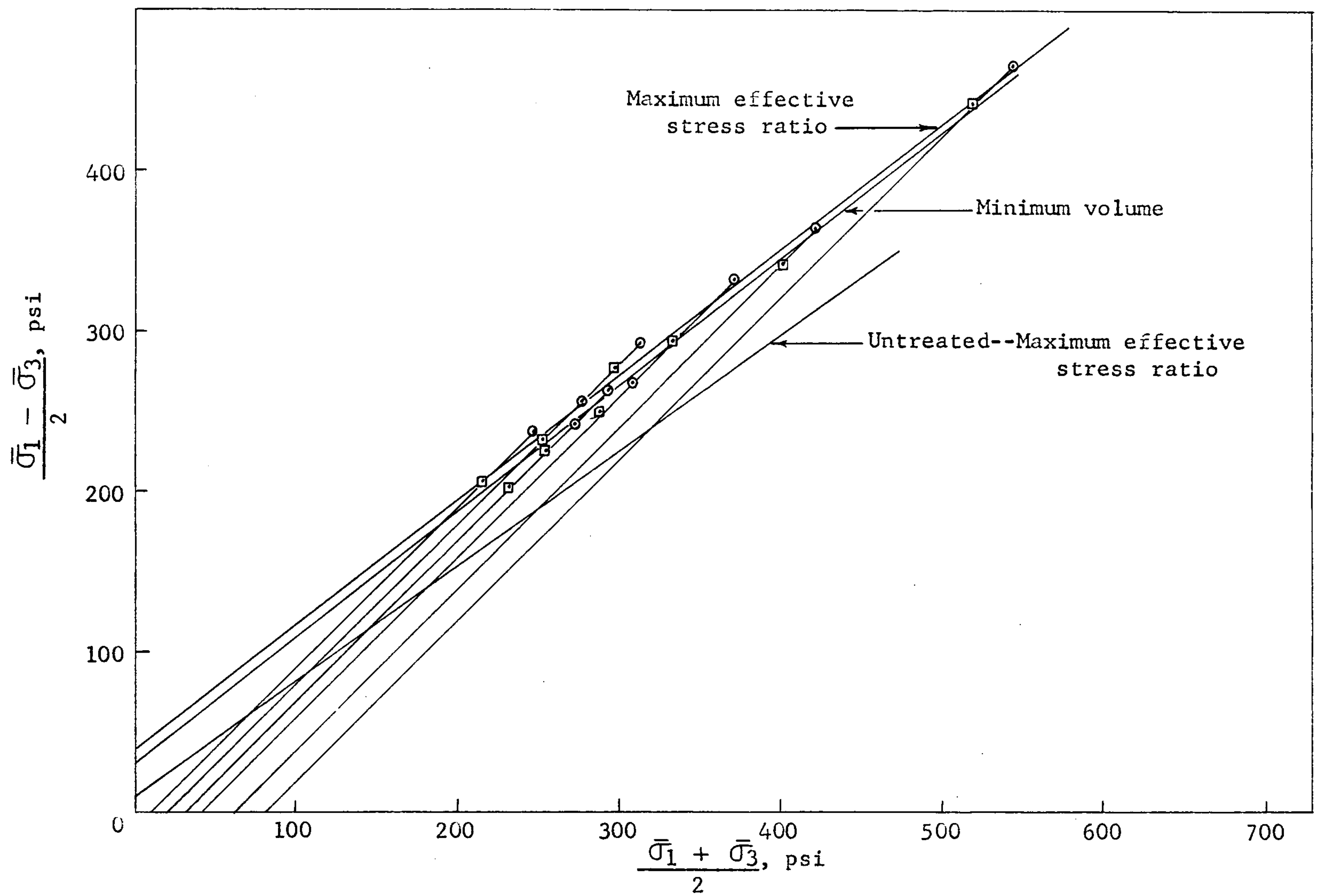

Figure 15. Modified Mohr-Coulomb diagram for Gilmore, 3% cement treatment, 28-day cure with envelopes for maximum effective stress ratio and minimum volume failure criteria.

A certain amount of sample variation will be noted in the figures, but most points fall on or near the envelope of failure.

Analysis of cement treated Garner stone was somewhat difficult due to a large degree of specimen variation. The variation is more pronounced at the point of minimum volume than it is at the maximum effective stress ratio and may be due to the method of testing. Rate of stress increase was very rapid to the point of minimum volume, and it is doubtful that the condition of minimum volume actually occurred at the exact instant the load reading was taken. To accurately determine the stress conditions at minimum volume of the Garner it would have been necessary to continuously measure the volume change.

Addition of 1% cement to the Garner crushed stone appears to alter the angle of internal friction as well as the cohesion; whereas 3% cement causes a large increase in cohesion with little change in the friction angle.

Cement treated Gilmore stone also had a large amount of sample variation, but not as pronounced as the Garner. Both cement contents appear to affect the cohesion and angle of internal friction of the Gilmore.

Shear strength parameters determined for the various conditions of cement content, and length of cure are presented in Table 6.

Table 6. Shear strength parameters determined by least squares method

Material and treatment	Failure criteria: Maximum effective stress ratio		Minimum volume	
	Ø', degrees	c', psi	Ø', degrees	c', psi
Bedford crushed stone:				
Untreated	45.7	6.7	46.2	4.2
1% cement 7-day cure	47.0	24.2	47.9	15.9
1% cement 28-day cure	44.6	42.5	45.5	29.6
3% cement 7-day cure	47.0	67.0	47.7	56.6
3% cement 28-day cure	45.3	78.7	46.0	70.5
Garner crushed stone:				
Untreated	49.2	14.2	49.5	5.6
1% cement 7-day cure	54.6	21.6	53.1	9.2
1% cement 28-day cure	49.0	41.2	46.3	30.4
3% cement 7-day cure	50.1	90.5	50.6	64.6
3% cement 28-day cure	51.0	96.2	51.2	87.9
Gilmore crushed stone:				
Untreated	45.1	17.1	45.5	8.9
1% cement 7-day cure	50.6	18.1	51.8	0.8
1% cement 28-day cure	51.2	18.2	51.5	3.2
3% cement 7-day cure	48.6	57.4	49.0	43.8
3% cement 28-day cure	50.6	64.0	51.1	52.3

Cohesion and angle of internal friction

General   Effect of cement treatment on the shear strength parameters of the three crushed stones can be more easily visualized by plotting cohesion with respect to the cement content as shown in Figure 16. Since only two cement contents were used, the points are connected with straight lines instead of smooth curves that may actually exist.

It can be seen that for the Bedford material the gain in cohesion between the 7-day and 28-day cure periods is relatively uniform for both cement contents.

Addition of 1% cement to the Garner stone appears to have little effect on cohesion after a cure period of 7 days. However, after a cure period of 28 days the cohesion is increased considerably. The addition of 3% cement to the Garner material causes a large increase in cohesion at 7 days cure and a further increase after 28 days cure.

Addition of 1% cement to the Gilmore stone has only minor affect on the cohesion even after 28 days of curing. Increasing the cement content to 3% results in increased cohesion, but of a lower magnitude than the other two stones.

The relationship between cohesion and cement content is not consistent for the three materials indicating the possibility of varying mechanisms of stabilization. The effect of the cement on the three crushed stones can be more clearly shown in Figures 17, 18, and 19. The plots have no special meaning other than showing the relationship between ϕ', c', per cent cement, length of cure, and the condition of failure together, instead of attempting to analyze them individually.

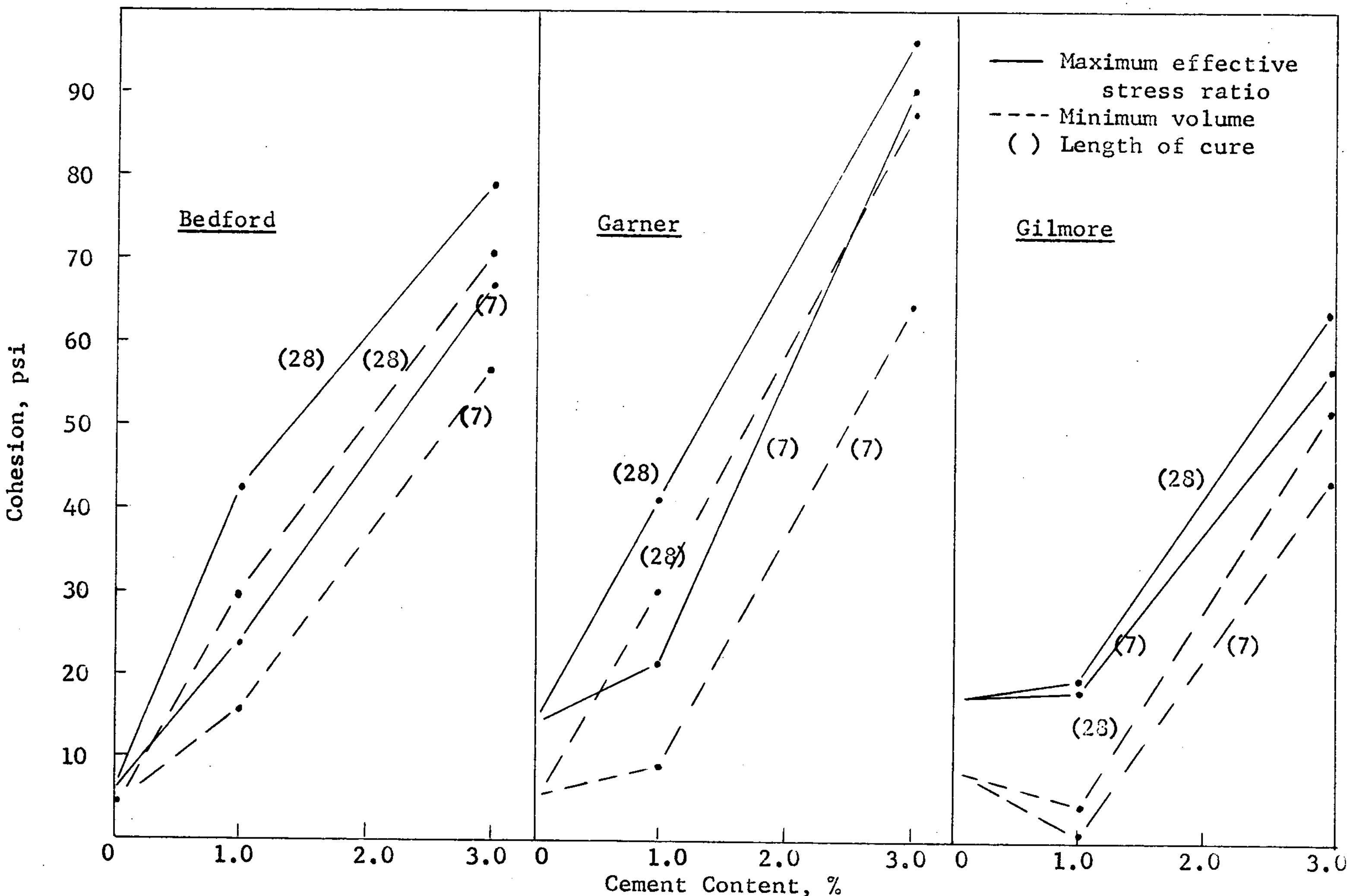

Figure 16. Cohesion-cement content relationship for the three crushed stones at maximum effective stress ratio and minimum volume criterion of failure

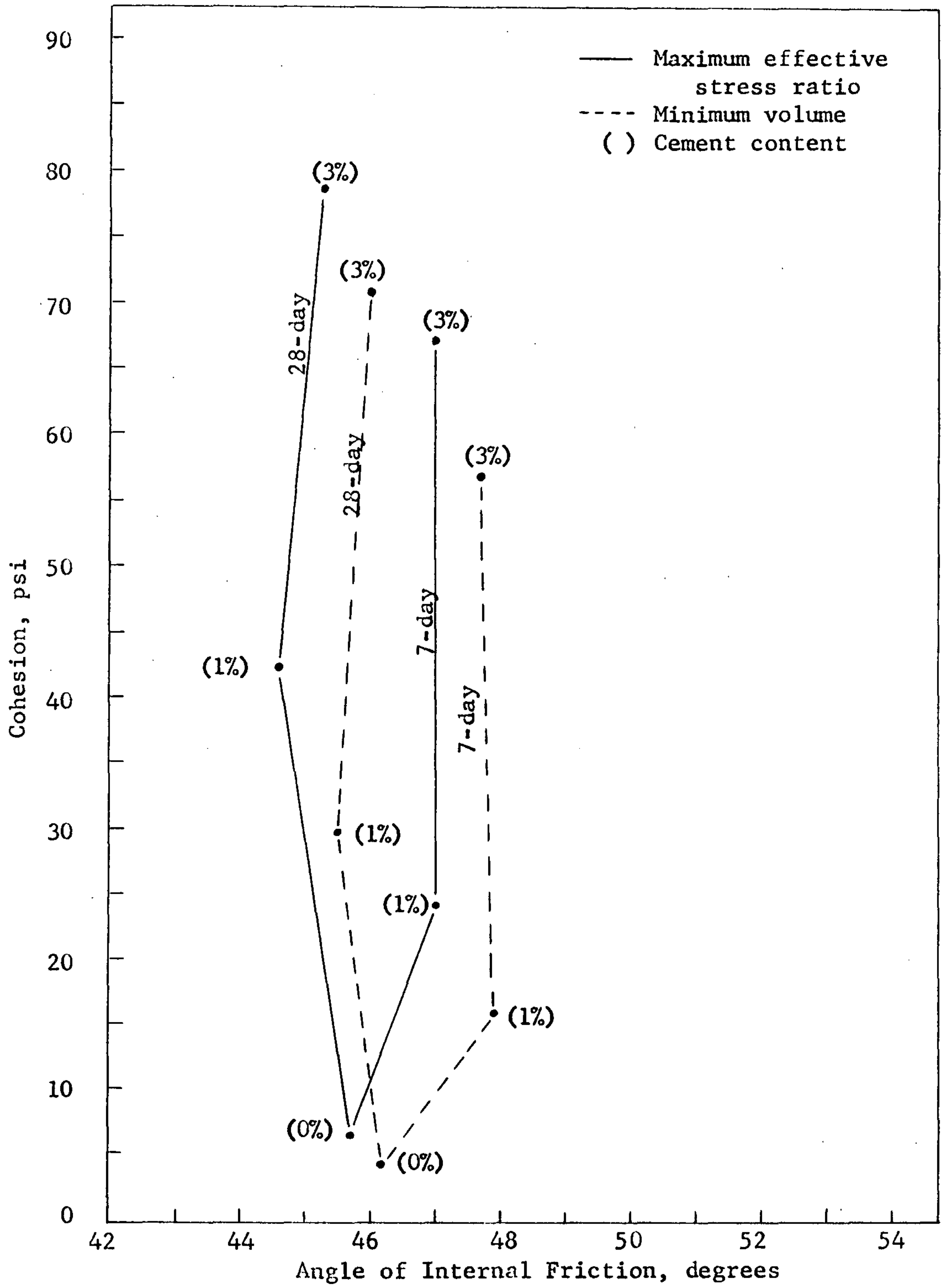

Figure 17. Effect of cement content and length of cure on shear strength parameters for Bedford crushed stone

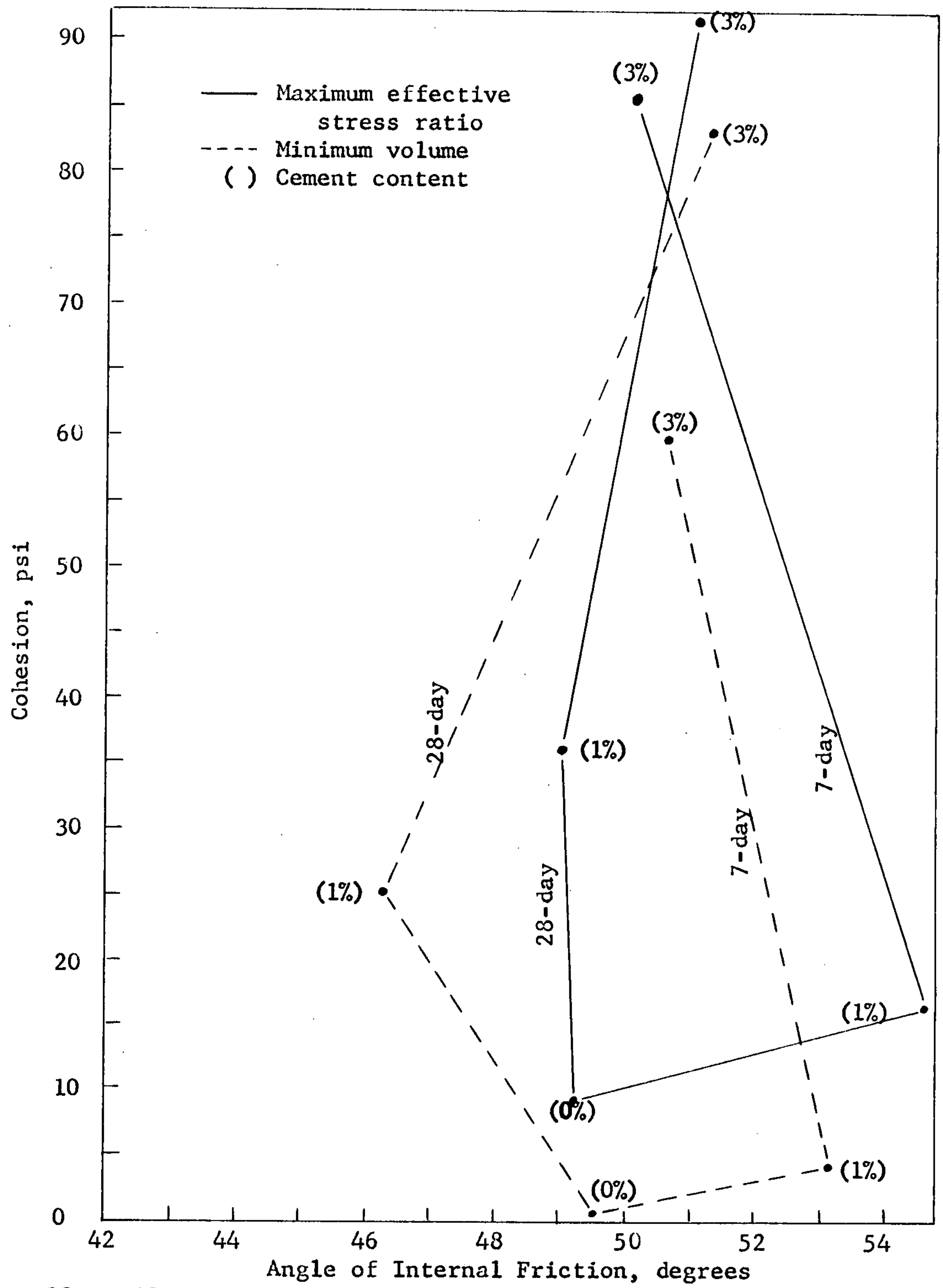

Figure 18. Effect of cement content and length of cure on shear strength parameters for Garner crushed stone

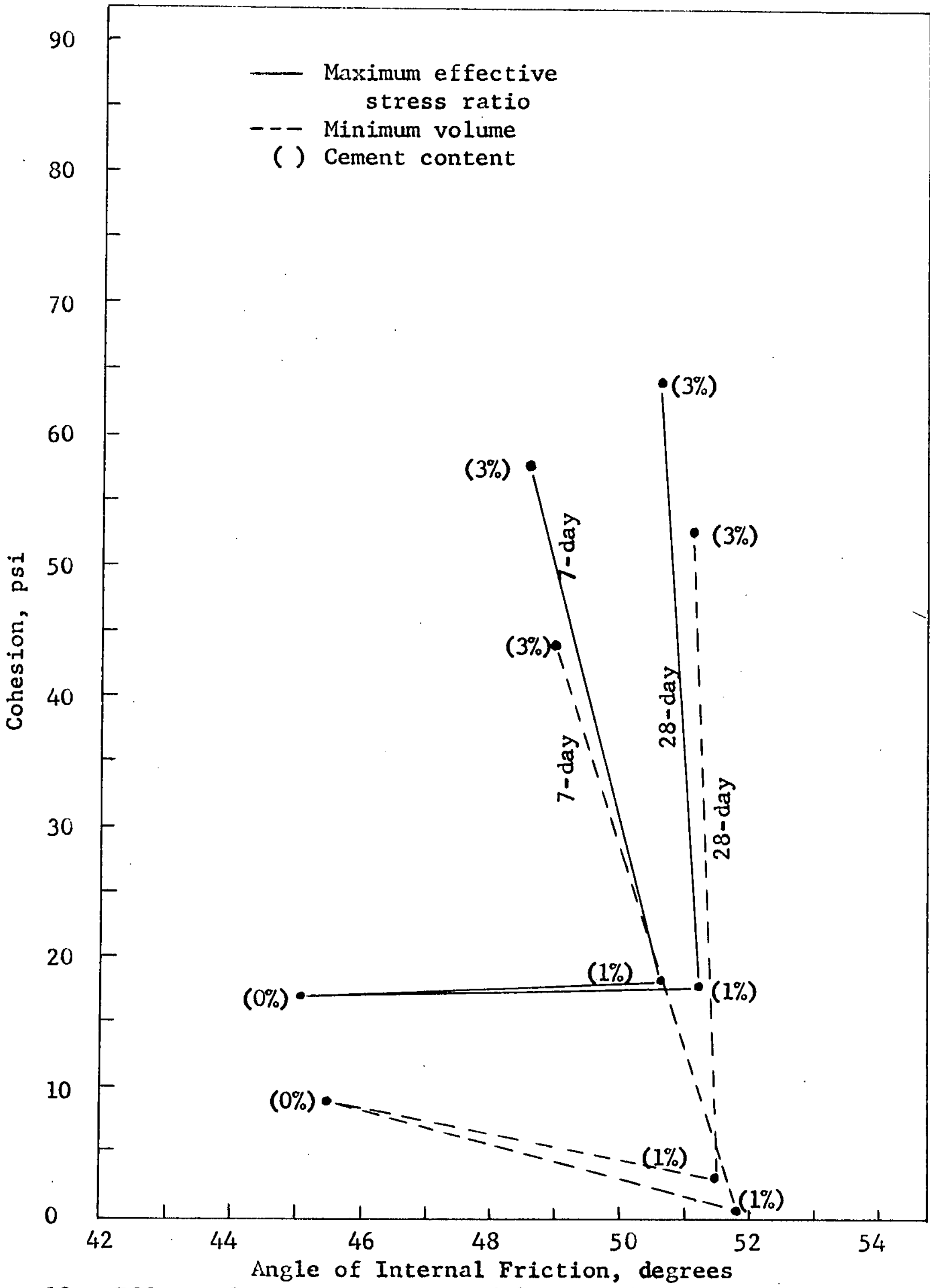

Figure 19. Effect of cement content and length of cure on shear strength parameters for Gilmore crushed stone

As mentioned previously, granular materials tend to exhibit the ability to resist shear through interlocking, and the change in shear resistance from conditions of minimum volume to maximum effective stress ratio may be an indication of the degree of interlocking. The effect of interlocking tends to decrease at higher lateral pressures (17). This can be shown by the fact that the difference between the stress conditions at minimum volume and at maximum effective stress ratio decreases, as the lateral pressure increases. This variation in interlocking results in a slight decrease in the friction angle, and an increase in cohesion between conditions of minimum volume and maximum effective stress ratio.

As may be noted from the data, it is difficult to determine the actual effect of the cement on the shear parameters of the materials. Not only are the properties of the materials altered by the cementing action, but also by variations in moisture content, density, and gradation, from that of the untreated materials. To determine the effect of the bonding action of the cement it would first be necessary to determine the properties of the cement treated materials at a time of zero cure. Since this is not practical, an attempt will be made to determine the changes in shear strength between cure periods of 7 and 28 days for each of the cement contents. Assuming that for a given material and cement content, the specimens are identical initially, the change in shear properties between 7 and 28 days should be due primarily to the increase in strength of the cement bonds.

Previous investigations into the effect of cement treatment on granular materials, have shown that cohesion increases with cement content, but that the angle of internal friction undergoes little change. The Bedford stone appears to follow this pattern. At seven day cure, both cement contents show an increase in cohesion with a small increase in Ø. At 28 day cure, the cohesion increases further, but there is a reduction in Ø from that obtained with the untreated stone. The results for both conditions of failure followed the same pattern.

<u>Bedford crushed stone</u> The Bedford stone is quite porous, and the texture of the surface is fairly rough enabling the formation of a strong cement bond between the aggregate and the matrix. The coarse aggregate is somewhat rounded in shape, and there is a higher percentage of fines than in the other two materials.

The change in stress conditions from minimum volume to maximum effective stress ratio, results in an increase in cohesion with a slight decrease in Ø for both the cement treated and untreated specimens, Figures 16 and 17. The magnitude of this change appears to be constant for the varying conditions of cement content and length of cure. Cement tends to increase the interlocking action of the untreated material by bonding the fines. Increasing the strength of these bonds, through increased length of cure or additional cement does not appear to increase the degree of interlocking. As the strength of the cement bond increases from 7 to 28 days there is an increase in cohesion with a reduction in Ø.

In summary, the addition of cement to the Bedford stone indicates that the cement bond tends to increase the cohesion, but has little effect on the shearing action within the material.

<u>Garner crushed stone</u> The Garner crushed stone treated with 1% cement at 7 days of cure has a large increase in Ø and a small increase in cohesion from that of the untreated material, Figure 18. After 28 days of cure, the cohesion is increased and Ø is reduced to a value lower than the untreated. At a cure period of 7 days, the 3% cement treated material shows a large increase in cohesion with a small increase in Ø from that of the untreated material, and additional curing resulted in further increases in both cohesion and angle of internal friction.

Visually the coarse aggregate of the Garner material has much the same shape and texture of the Bedford crushed stone. However, the Garner produces much higher densities than either of the other two stones, which is partially indicative of the presence of more points of grain to grain contact. The strength properties of any cement treated material are dependent upon the number of these contact points, as this is where cement bonds may develop. Uniform sand has relatively few points of contact and requires higher cement contents for adequate stabilization. As the gradation of a material becomes more beneficially distributed, the cement content required for adequate stabilization tends to decrease.

The variation in strength between individual specimens appeared to be more pronounced with the Garner crushed stone than was observed for the other two stones. Strength variation was not directly related to variations in density but may have been related to uneven distribution of cement within the specimen or some other form of sample variation. It was evident that the addition of cement had a much greater effect on the shear strength parameters of the Garner crushed stone than either of the other crushed stones and thus, the variations in individual specimens would be more pronounced.

The change in shear strength between the failure conditions of minimum volume and maximum effective stress ratio for the 1% cement treated Garner does not follow the same pattern as the Bedford and Gilmore materials. Between these points there is an increase in both ∅ and c. The fact that the angle of internal friction increases between these points cannot be explained by the information available.

The addition of 3% cement to the Garner crushed stone tends to increase interlocking as indicated by the high increase in cohesion and a slight decrease in ∅ from conditions at minimum volume to maximum effective stress ratio. The change in strength properties between 7 and 28 days cure, due to the increase in the strength of the cement bond, results in an increase in cohesion and an increase in the angle of internal friction.

Gilmore crushed stone Gilmore stone did not react in the same manner as the Bedford or Garner stones, Figures 16 and 19. At the point of maximum effective stress ratio there was an increase in ∅ and c for both cement contents at 7 day cure. From 7 to 28 days cure, the cohesion of the 1% cement treated material reduced slightly and had a fairly large increase in ∅, while the 3% material had an increase in both ∅ and c.

The Gilmore stone is a very hard, angular material having the smallest amount of fines of the three stones, Table 4. Untreated Gilmore specimens had a much greater tendency to collapse, when handled, than specimens of the other two stones, though produced a higher amount of cohesion, Table 6. The larger value of cohesion may be due to a higher degree of interlocking that the material can develop, as is indicated by the increase between the two conditions of failure, Figure 16, particularly at 0 and 1% cement contents.

It appears that cement may not function as just a bonding agent at points of contact between the larger Gilmore aggregate and the matrix as it does with the Bedford stone. Instead the cement tends to bond the fines together resulting in a matched or interlocked coarse material that develops its strength from the interlocking rather than the bonds between the aggregate. To better illustrate this point, shear strength of a material composed of uniform spheres can be increased through the addition of smaller spheres which tend to fill the voids between the larger spheres and increase the effect of interlocking. The more rigid the material in the voids can be made, the higher the degree of interlocking. The same is true for angular material, however it is capable of developing a higher degree of interlocking due to particle shape. The Gilmore stone is very angular resulting in very irregular shaped voids. The cement may tend to strengthen the fines present in the voids between the coarse aggregate and create rigid, coarser particles, matching the shape of the voids.

The method of strength increase mentioned above can also be shown by the strength properties of the 1% cement treated Gilmore material at the point of minimum volume, Figure 19. The cohesion is reduced from 8.9 psi for the untreated material, to 4.8 psi and 4.7 psi for the 7 and 28 day cure periods respectively. The angle of internal friction is increased from 45.5 for the untreated material to 51.8 for the 7-day cure and 51.5 for the 28-day cure.

The degree of interlocking as indicated by the increase of cohesion between minimum volume and maximum effective stress ratio is quite large as shown by the cohesion increase with a small decrease Ø, Figure 19.

The addition of 1% cement apparently does not result in bonding of the aggregate but results in bonding of the fines, increasing the angle of friction. Additional cement causes no further increase in Ø but results in higher cohesion.

Pore Pressure

Pore water pressures that develop in soil during loading are indicative of the tendency for a saturated soil structure to change volume with strain; i.e., negative pore pressure indicates expansion, while positive pore pressure indicates contraction. This condition is only valid when conditions of stauration or near saturation exist. A decrease in the degree of saturation will result in a decrease in the magnitude of pore pressure developed for equal amounts of volume change due to compression of air in the voids.

Figures 20 through 23 show the relationship of pore pressure to lateral pressure at both conditions of failure; i.e., maximum effective stress ratio and minimum volume. Irregularities can be attributed to variations in the degree of saturation. The difference between each pair

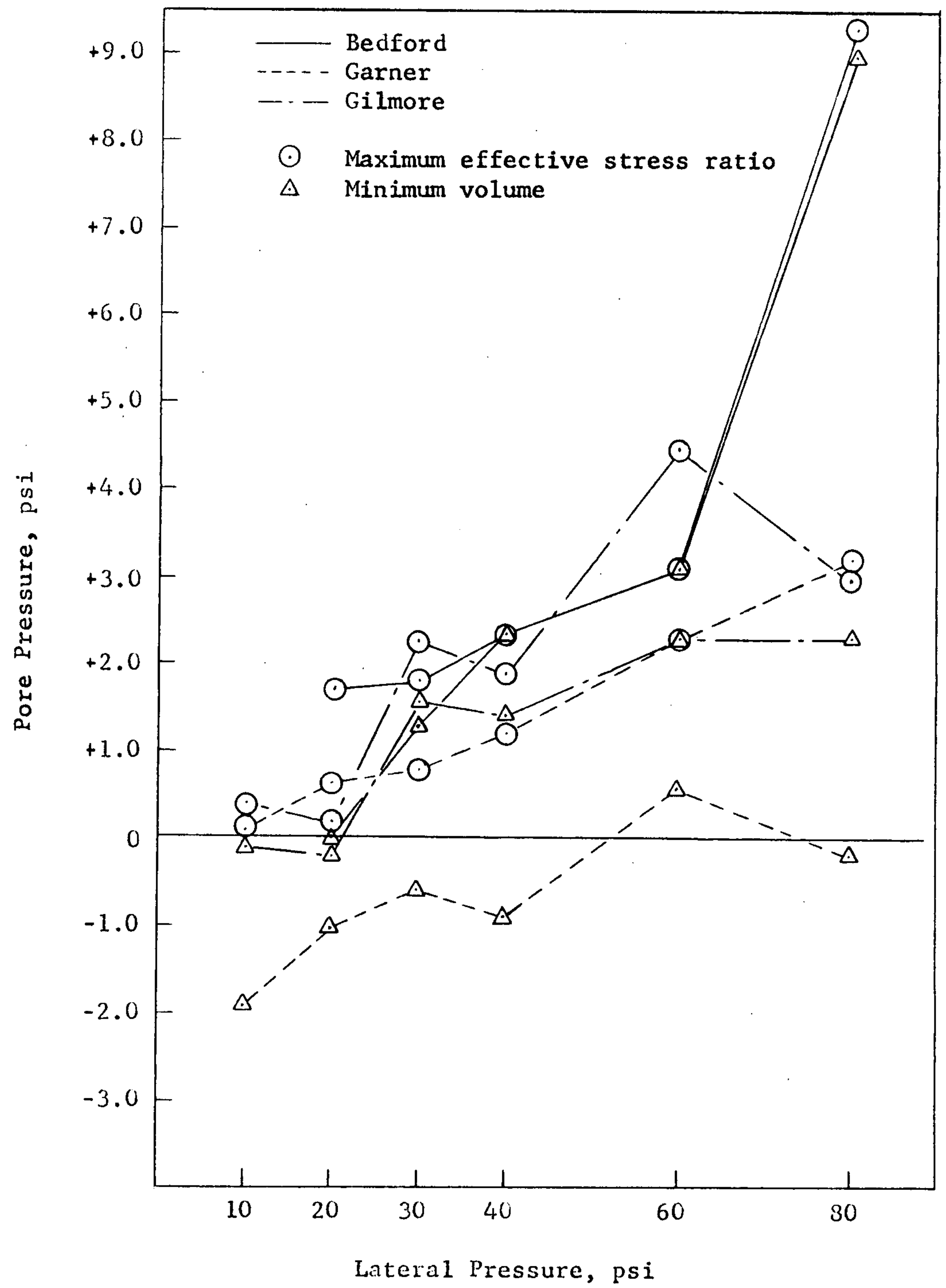

Figure 20. Pore Pressure-lateral pressure relationship for the three untreated-crushed stones

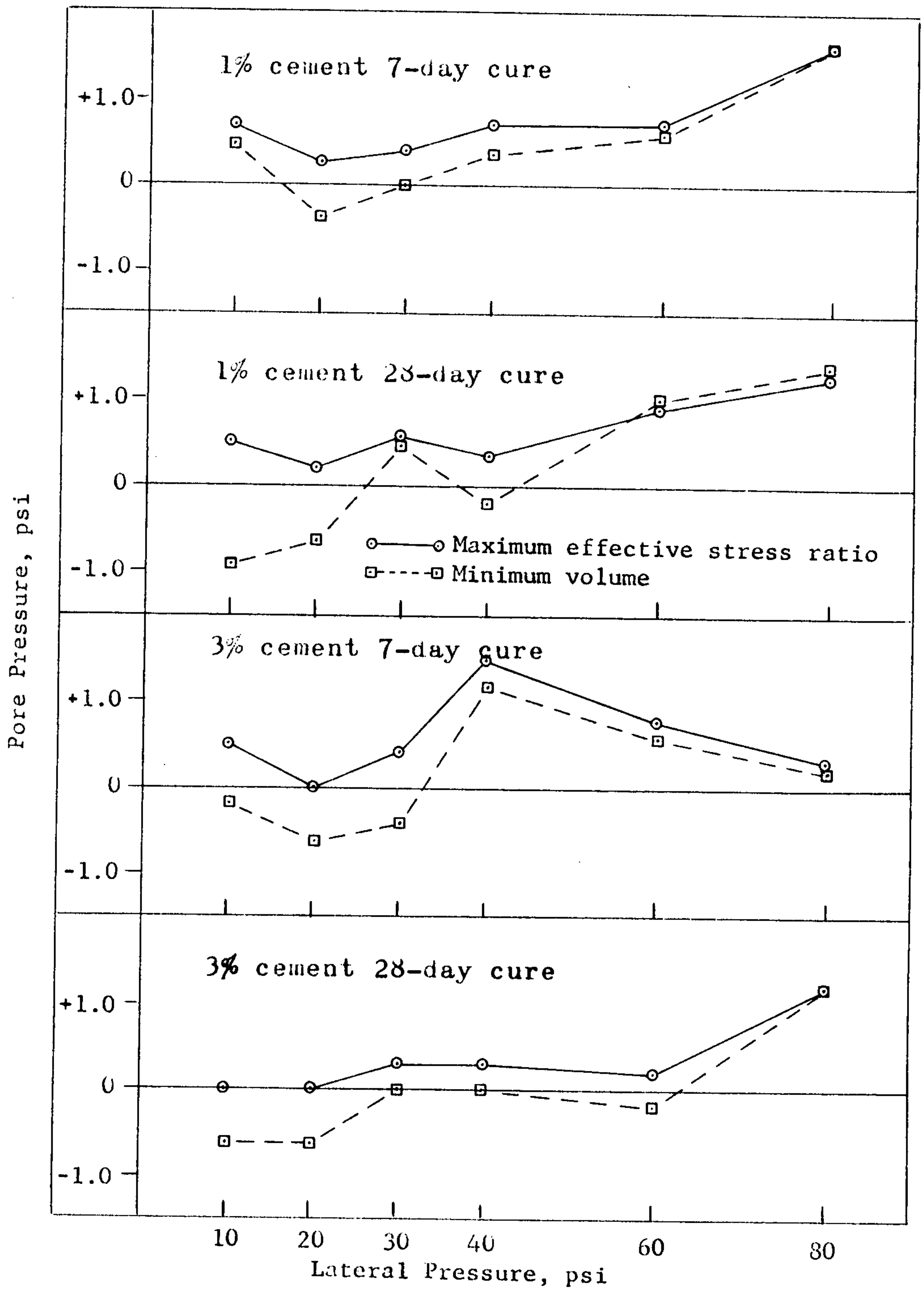

Figure 21. Pore pressure-lateral pressure relationship for the cement treated Bedford crushed stone

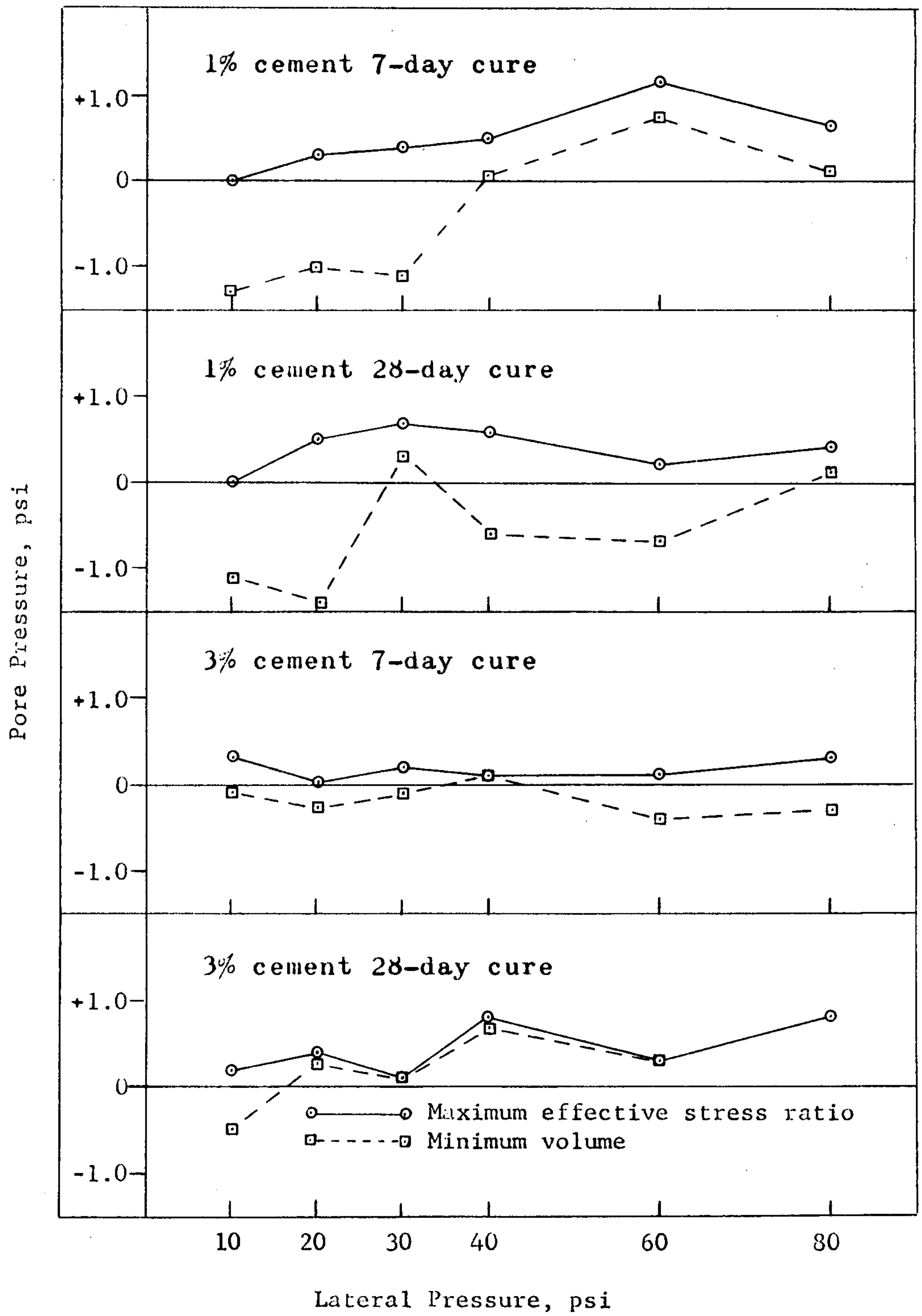

Figure 22. Pore pressure-lateral pressure relationship for the cement treated Garner crushed stone

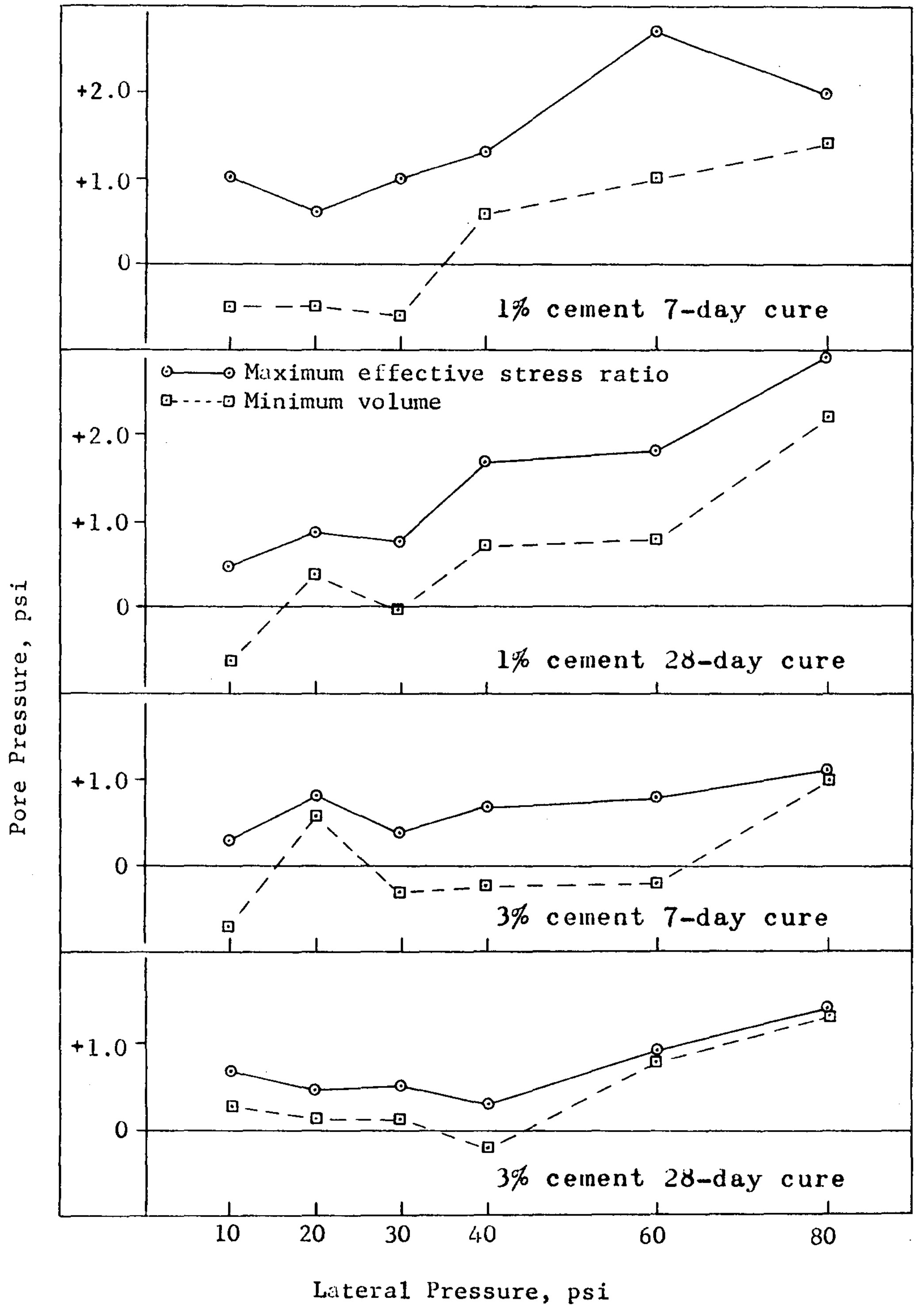

Figure 23. Pore pressure-lateral pressure relationship for the cement treated Gilmore crushed stone

of curves is an indication of the amount of expansion required to develop the stress conditions at maximum effective stress ratio. At the lower lateral pressures, the difference is quite large, but tends to decrease with increasing lateral pressure and can be attributed to the greater amount of initial (consolidating) volume decrease at the higher lateral pressures.

Increase in cement content generally resulted in lowering of pore pressures at minimum volume, and less expansion was required to reach the maximum effective stress ratio state. Comparison of Figures 21, 22, and 23 with Figure 20 shows the obvious reduction in pore pressures due to cement treatment of the three crushed stones. However, reduction in pore pressure was much greater for the Bedford than for either the Garner or Gilmore materials. Cement probably reduces the plasticity of the fines in the Bedford and in turn reduces the tendency for volume decrease.

Strain

The amount of strain required to attain the failure conditions of minimum volume and maximum effective stress ratio are shown in Figures 24 through 27.

Addition of cement to a soil tends to form a brittle material; that is, the point of ultimate strength occurs within smaller increments of strain than for the untreated material. Increases in cement content

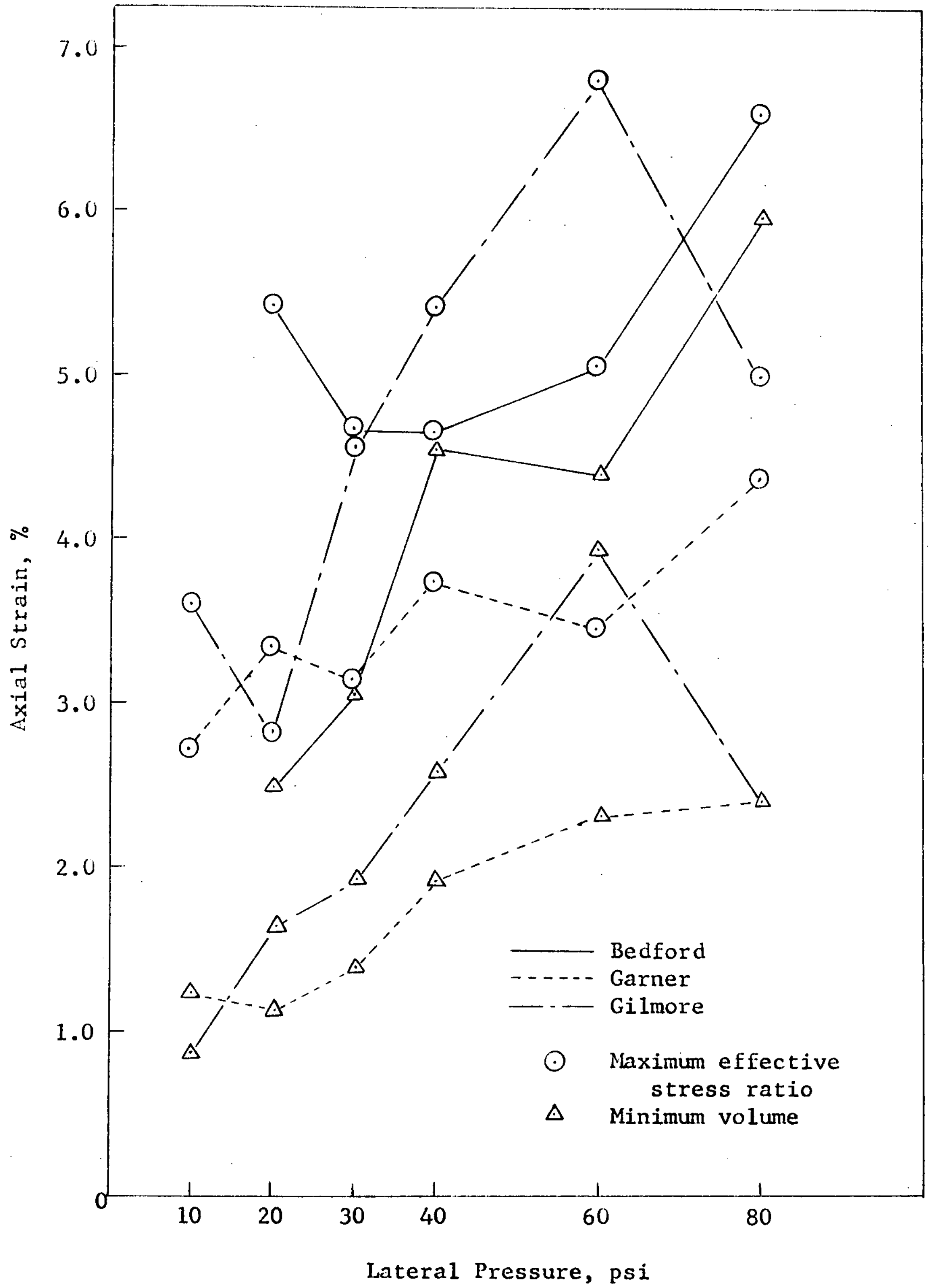

Figure 24. Axial strain-lateral pressure relationship for the three untreated crushed stones

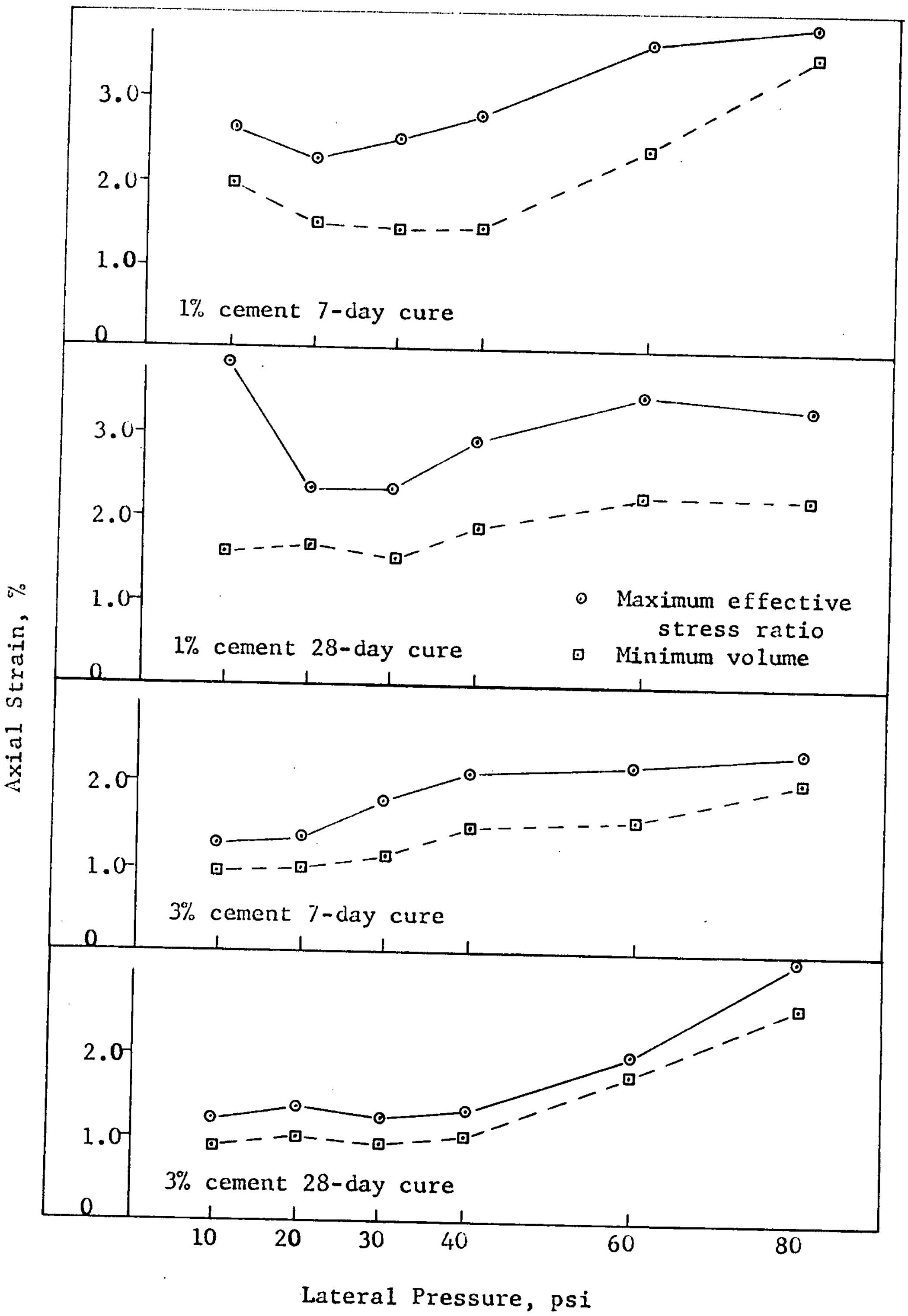

Figure 25. Axial strain-lateral pressure relationship for the cement treated Bedford crushed stone

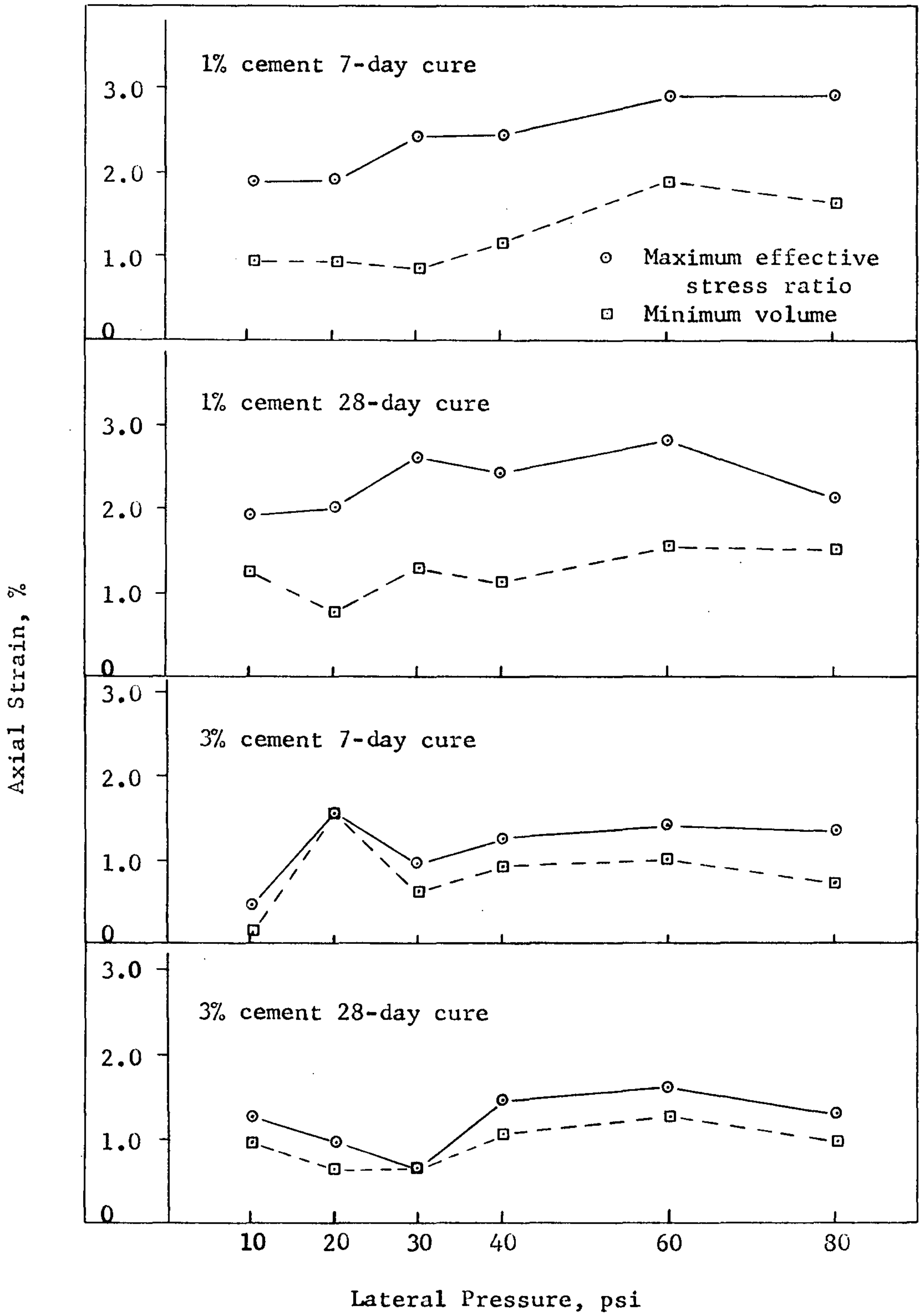

Figure 26. Axial strain-lateral pressure relationship for the cement treated Garner crushed stone

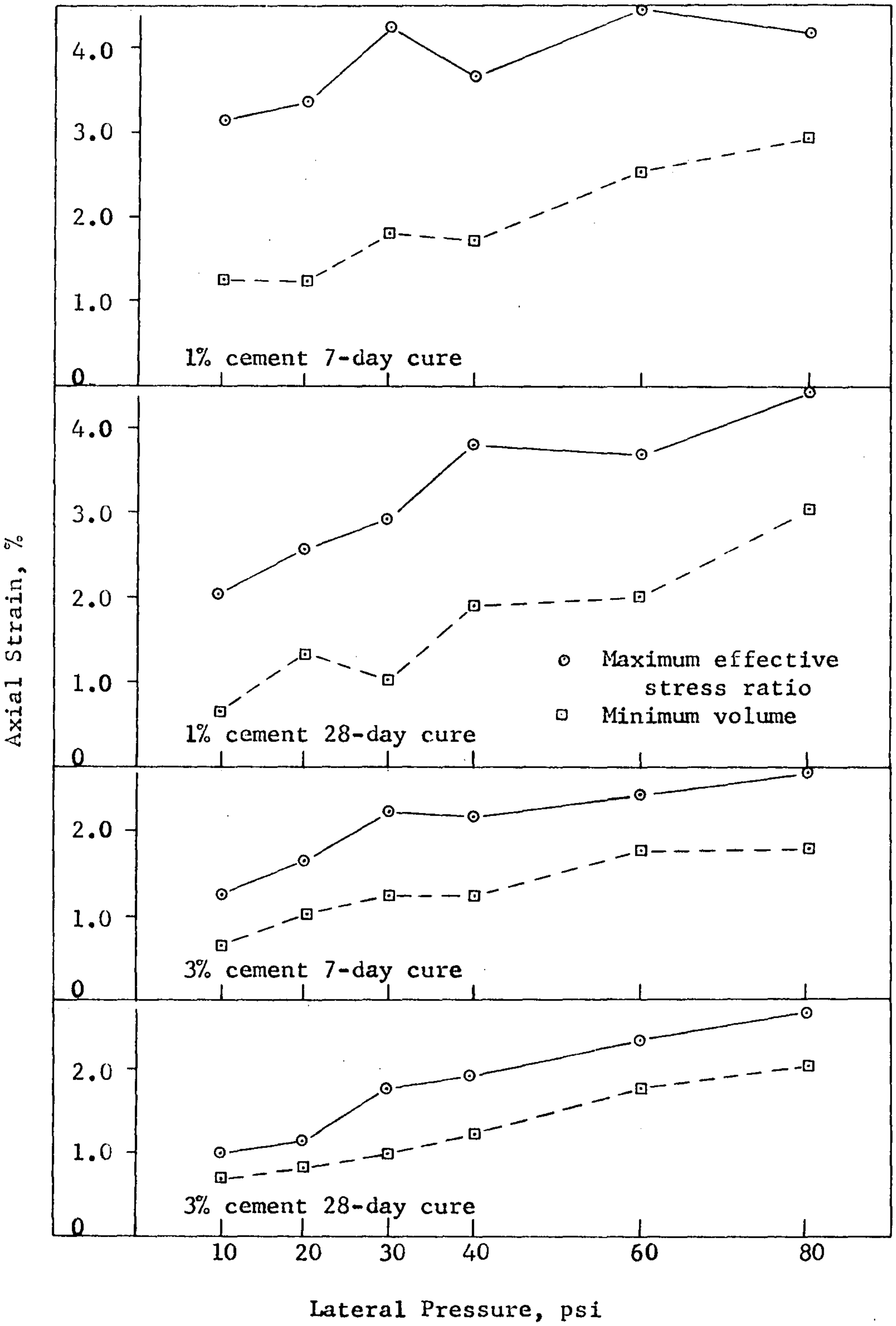

Figure 27. Axial strain-lateral pressure relationship for the cement treated Gilmore crushed stone

normally result in a corresponding decrease in the amount of strain required to reach ultimate strength.

It can be seen from Figure 24 that the variance of strain between the conditions of minimum volume and maximum effective stress ratio is quite pronounced for the untreated material. Also the amount of strain required to achieve these conditions generally tends to increase with increasing lateral pressure.

Addition of cement to the three crushed stones tends to reduce the amount of strain required to achieve conditions of minimum volume and maximum effective stress ratio, Figures 25, 26 and 27. Increases in strength through increases in the amount of cement, or length of cure, results in a corresponding decrease in strain. The effect of lateral pressure on the strain is not as pronounced for the cement treated material as for the untreated material. This is more evident for the Garner crushed stone than for the other two materials.

As mentioned previously, between the conditions of minimum volume and maximum effect stress ratio, the specimen begins to expand which may result in disruption of the cement bond. Thus, as the portion of the strength due to the cementing action within a specimen is increased, due to increased cement content, or curing, there is a corresponding decrease in the amount of strain that can be tolerated between the conditions of minimum volume and maximum effective stress ratio.

Volume Change

The initial portion of the analysis of results was based on current methods of analysis of shear strength. It was felt that these forms of analysis did not satisfactorily indicate the mechanism of failure. Use of a different concept of failure in the analysis of results, has indicated that shear strength only, as a means of evaluation of the overall stability of granular material, may result in values that are unique only to the method of testing, and which do not actually occur under field conditions.

Evidence for this belief is suggested by the relationship between the major principal stress and volume change during initial phase of axial loading, Figure 28. With application of axial load for a given lateral pressure, the volume of the specimen tends to decrease, occurring almost entirely in the vertical direction. The specimen then reaches a point of minimum volume decrease after which the volume begins to increase with additional increments of strain. This volume increase must be entirely in the horizontal direction. During the initial portion of the expansion phase, the major principal stress ratio continues to increase until a point of maximum effective stress ratio is reached. As many investigators have indicated, this expansion is required to overcome interlocking and allow for the formation of a failure plane.

It is felt by the authors, that this mode of failure develops only under conditions of constant lateral pressure such as in the triaxial shear test and that such conditions may not occur in the field since lateral pressures will increase as a result of resistance to expansion of the loaded material until a condition of limiting lateral support is achieved. At this point, the maximum lateral support is developed and the material fails by shearing as in the triaxial shear test.

Under field conditions this limiting value may be dependent upon the amount of restraint given by the shoulders and the surcharge adjacent to the point of loading, as well as the materials being utilized.

The above mentioned form of stability is illustrated by the relationship between the major principal stress and percent volume change, Figure 28. Assume that a low lateral pressure exists in a base course material prior to the application of an axial load. As the load is applied, the base course material will deflect vertically downward, until a point of minimum volume is achieved. After achieving this point, horizontal expansion increases rapidly resulting in increased lateral support and increased bearing capacity. This progressive increase in lateral support will continue until a limiting value of lateral support is achieved. This tends to indicate that the stability of a granular material is not entirely a function of the shear strength, but must also be a function of the lateral support that can be developed, and of the expansion required to develop that lateral support.

Another manner for the reader to visualize the above illustration is to assume an imaginary line tangential to the curves of Figure 28, beginning at zero volume change and moving up to the left towards about 700 psi effective stress. The points of minimum volume for each lateral pressure condition are close to this line. As the axial load is applied, at a low lateral pressure, the stress increases to the point of minimum volume, lateral expansion starts, confining pressure increases and the process is repeated until a limiting value of confinement (dependent on restraint of shoulder, surcharge and type of material) is achieved.

It is thus felt that the mode of failure in a base course is by progressive build-up of lateral support by lateral expansion of the loaded

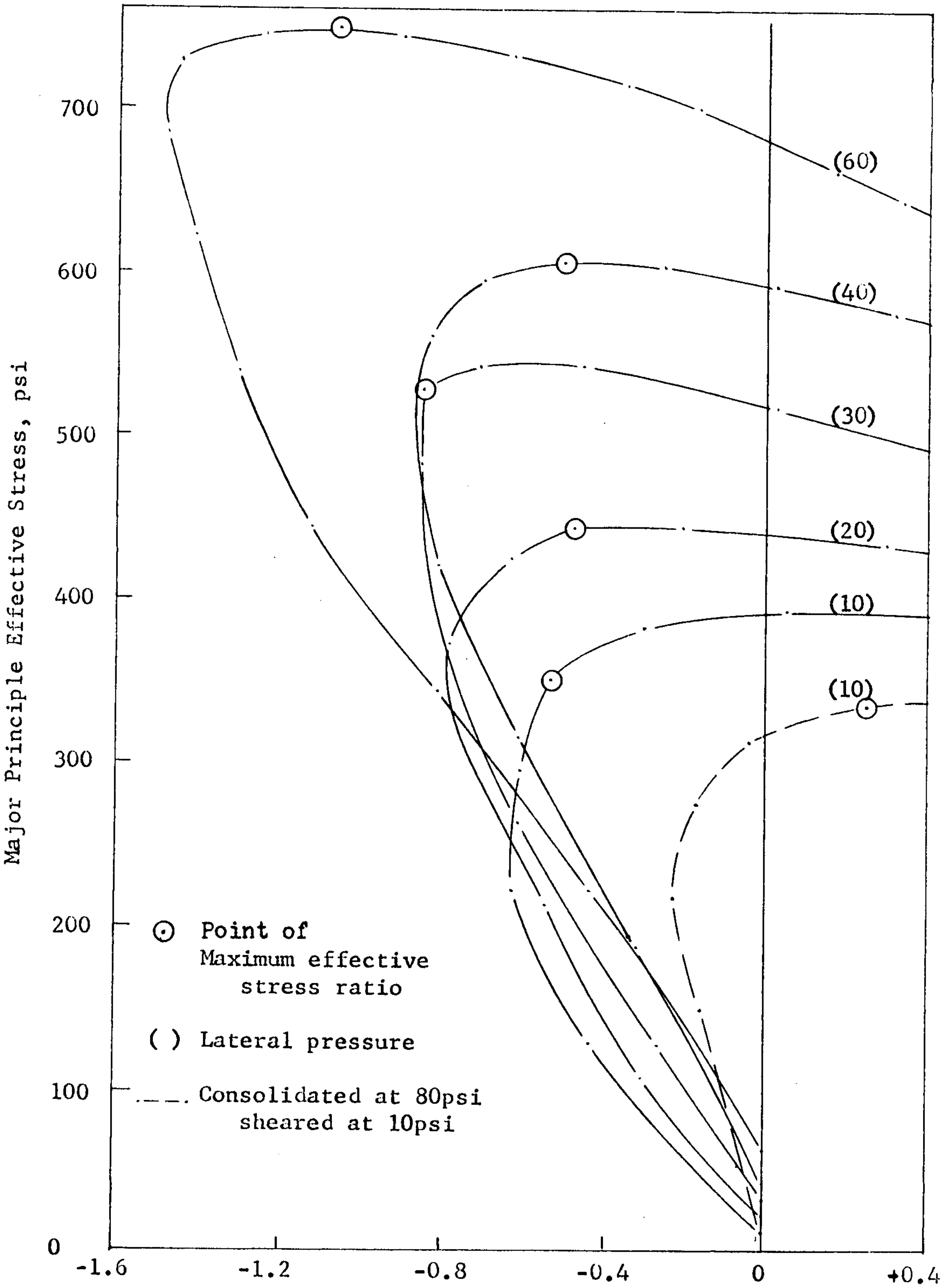

Figure.28. Major principle effective stress versus volume change for Bedford, 3% cement treatment, 7-day cure

material. Prior to lateral expansion, the strength properties may be that of the laboratory tested material, but after lateral expansion occurs, the strength properties of a given core of material are dependent upon the surrounding material.

Initial compression under a small increment of strain has been refered to as elastic compression because the elastic Poisson's ratio is less than one-half (26). As strain increases, expansion predominates, because the plastic Poisson's ratio may be greater than one-half (26). Reaction of the various specimens under load, with respect to volume change and axial strain, is shown in Figures 29 through 34. Initial slope of the curves shown, may be assumed to represent a degree of magnitude of Poisson's ratio. Since Poisson's ratio is defined as the ratio of lateral to vertical strain under axial loads, it can be shown that when lateral strain equals zero, volume change is equal to the axial strain and the material is in a compressed state. Likewise, for a non-compressible material, for which Poisson's ratio is about 0.5, both the lateral and vertical strains are finite quantities and the volume change is near zero.

It may be seen in Figures 29 through 34, that cement treatment of the three granular materials shifts the axial strain-volume change curves closer to the condition of zero lateral strain than with the untreated materials. The failure point of minimum volume is also much closer to this line for cement treated than for untreated materials. Thus, it can be seen that the amounts of both lateral and vertical strains developed in a treated specimen during axial loading may generally be reduced as compared to the untreated materials, up to the point of failure.

For the untreated materials, the slope of the volume change-strain curves is much closer to the condition of Poisson's ratio equal to 0.5,

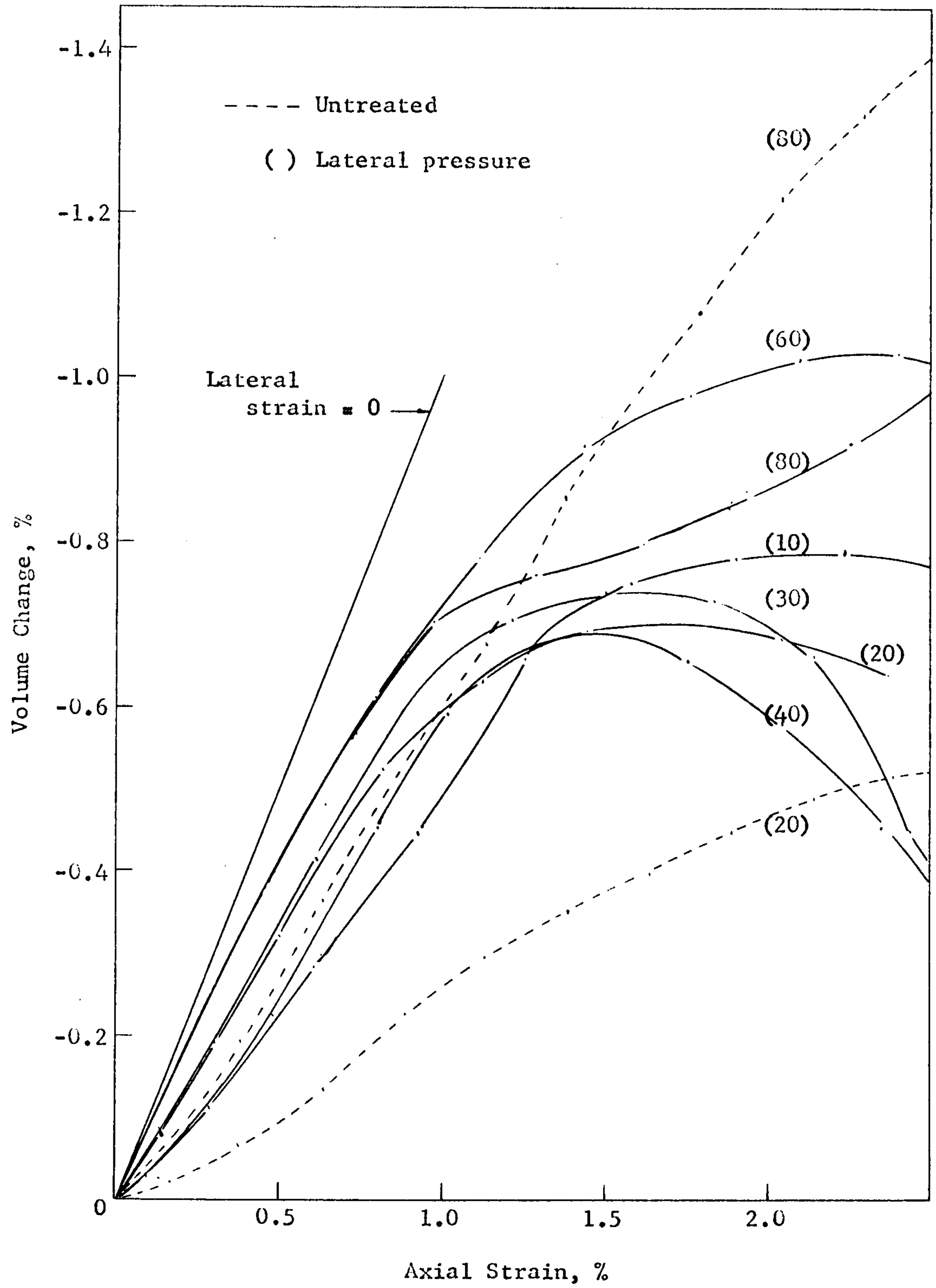

Figure 29. Volume change-axial strain relationship for Bedford, 1% cement treatment, 7-day cure

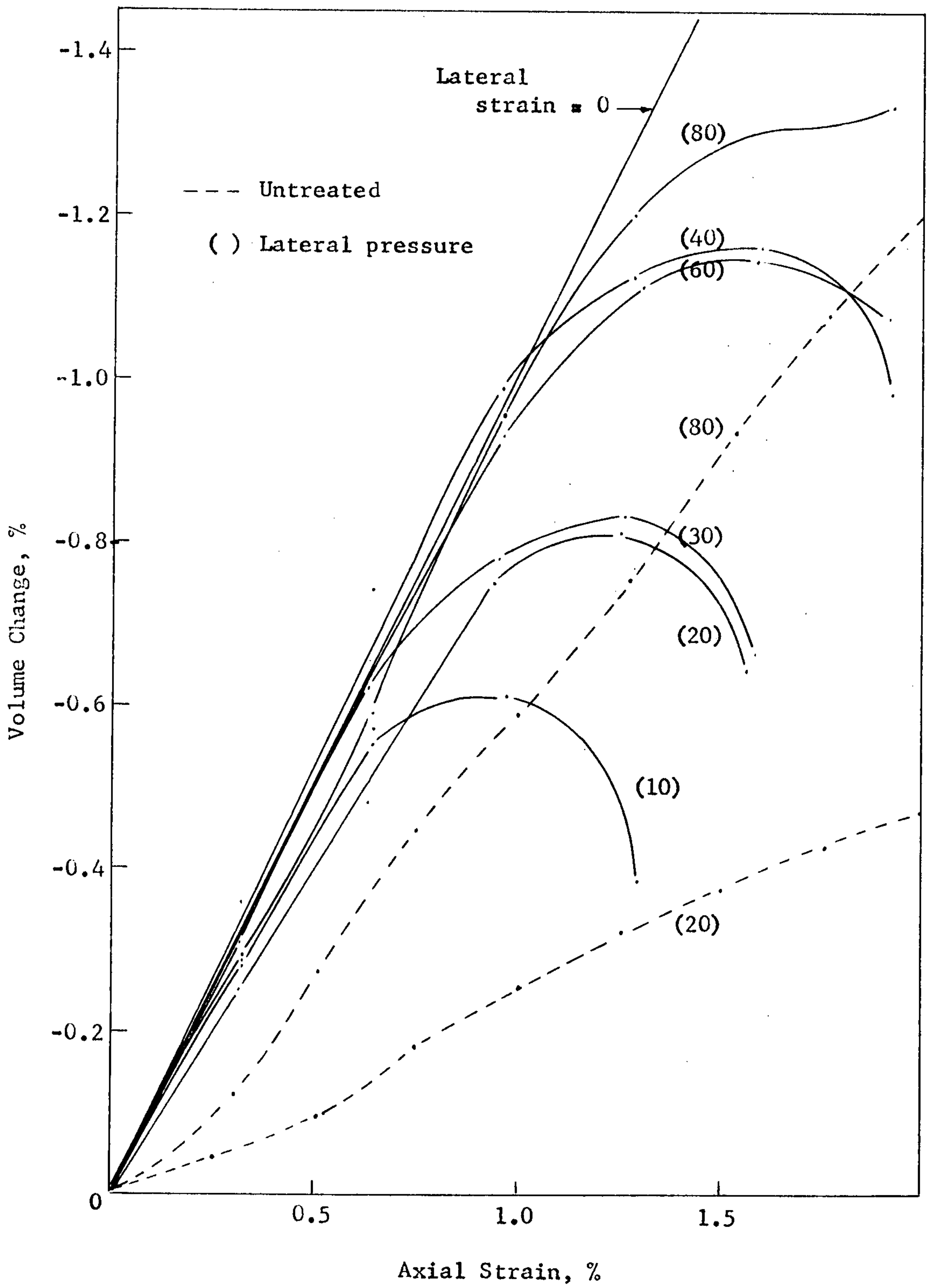

Figure 30. Volume change-axial strain relationship for Bedford, 3% cement treatment, 7-day cure

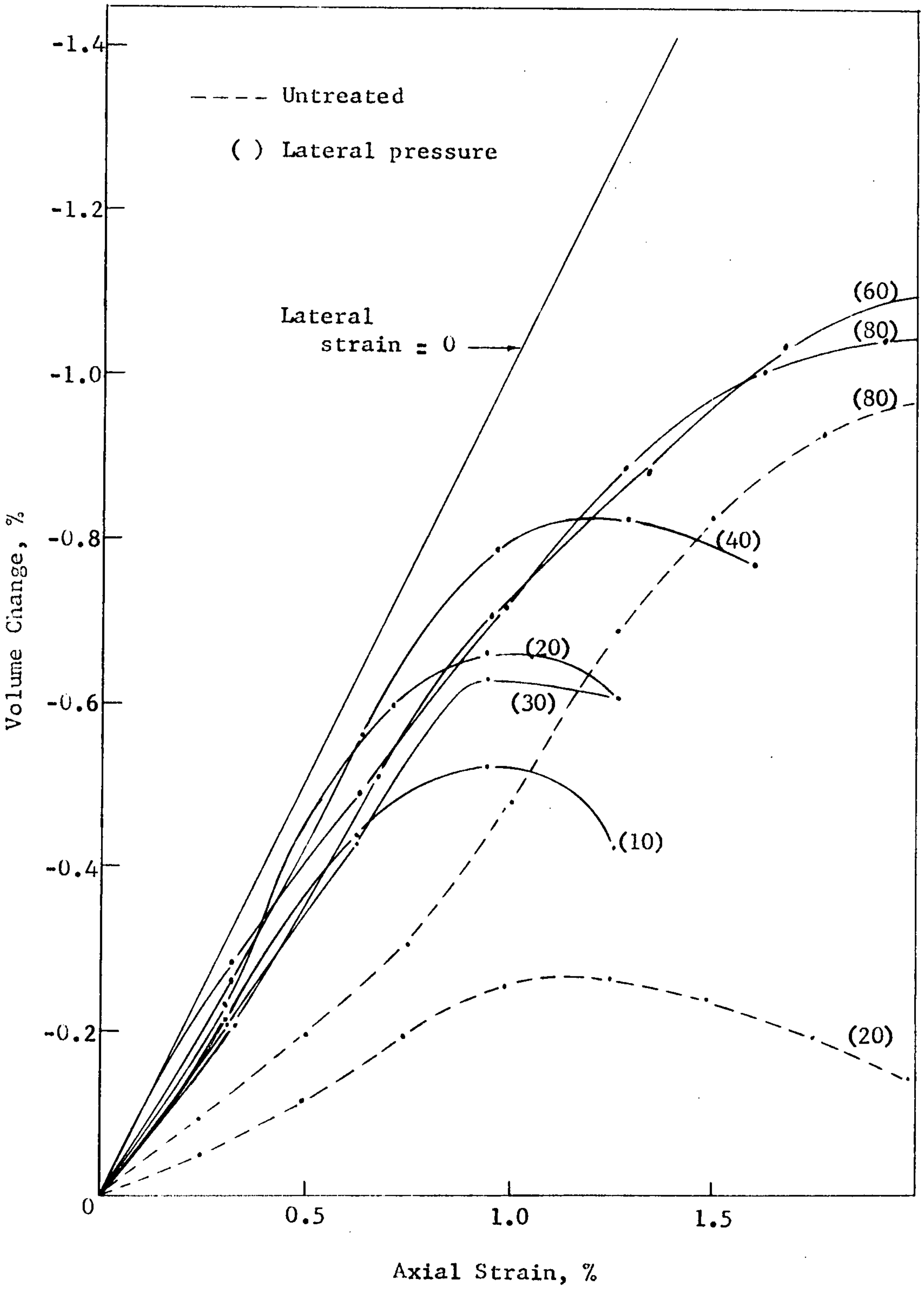

Figure 31. Volume change-axial strain relationship for Garner, 1% cement treatment, 7-day cure

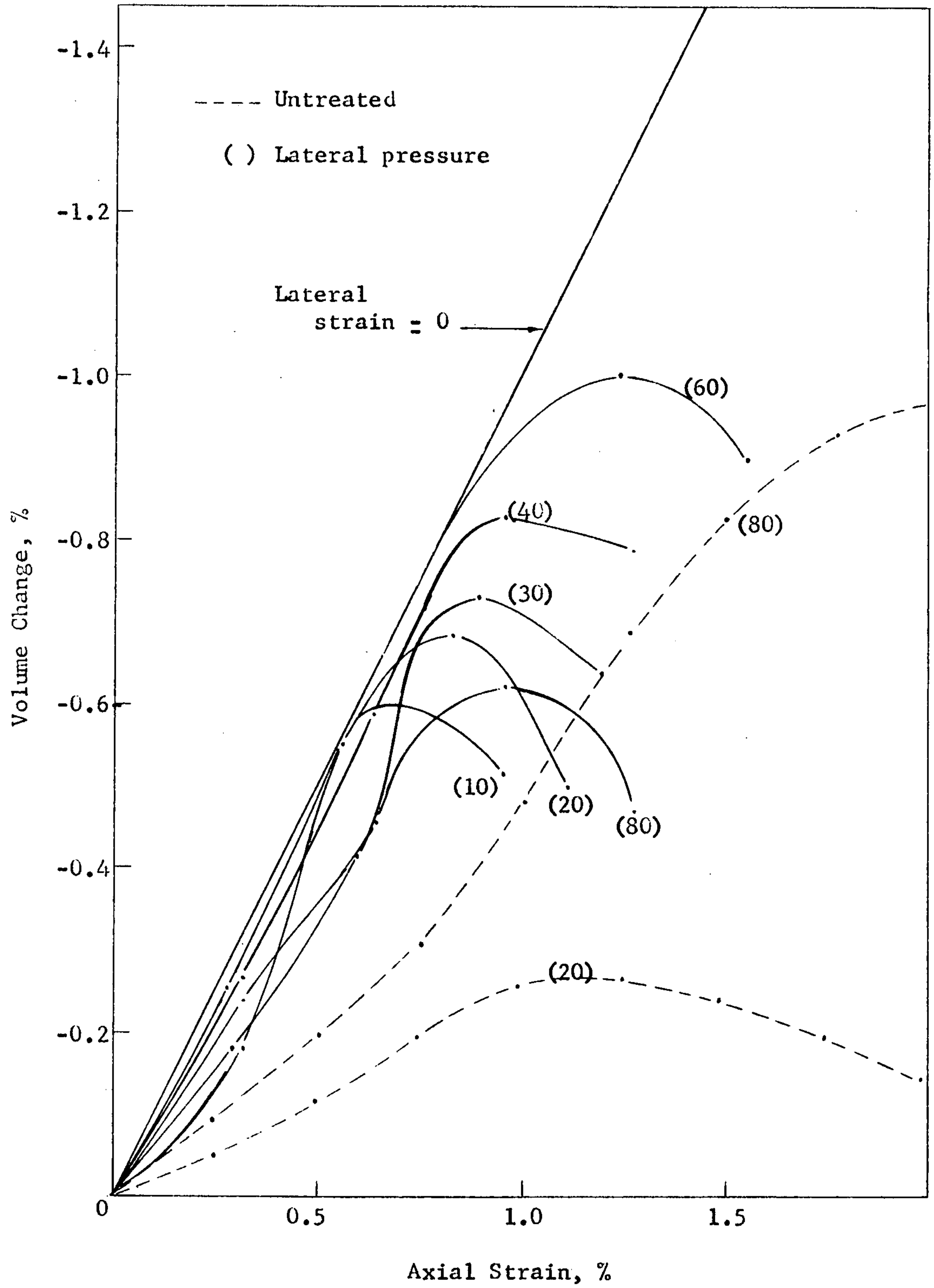

Figure 32. Volume change-axial strain relationship for Garner, 3% cement treatment, 7-day cure

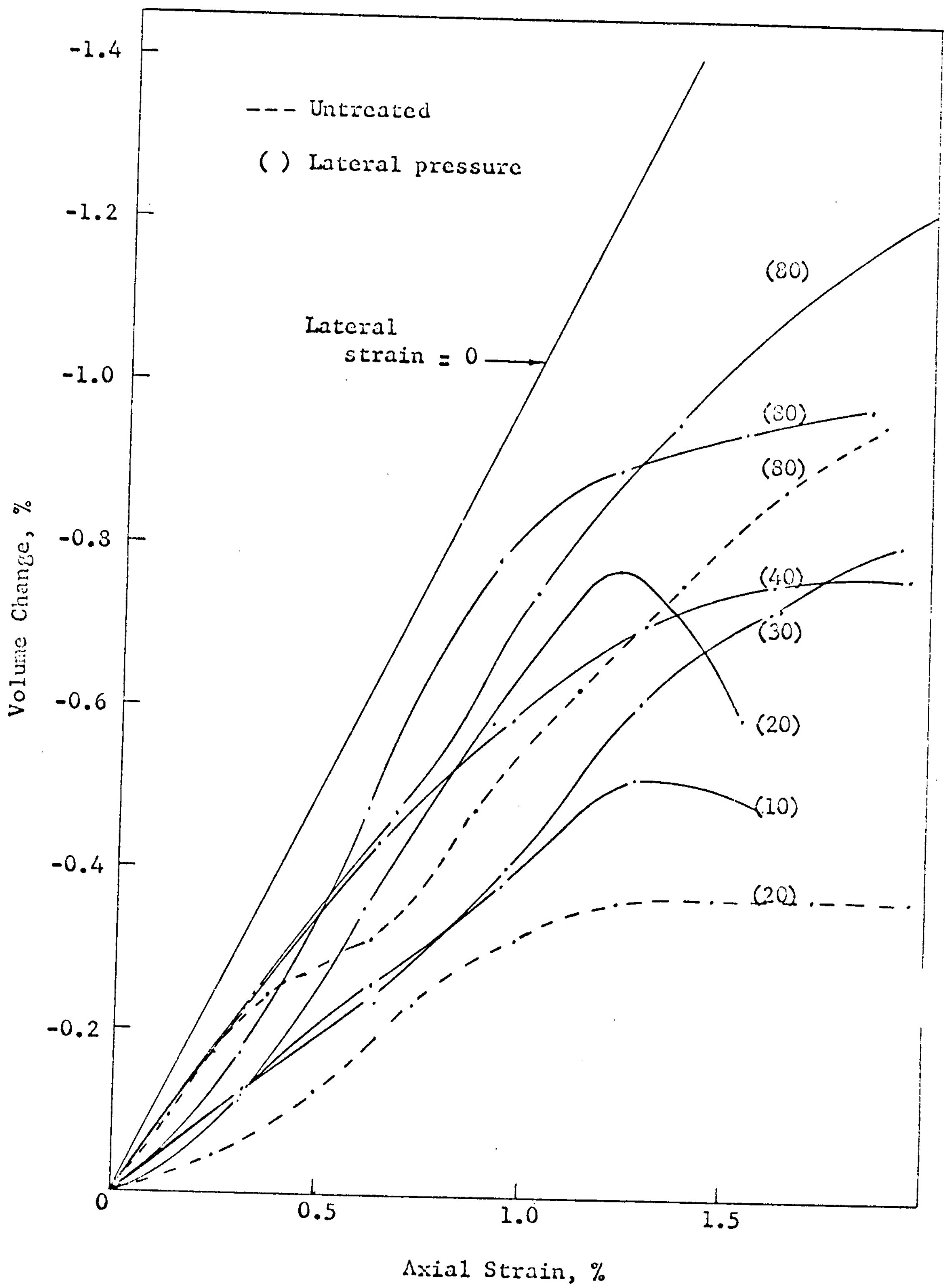

Figure 33. Volume change-axial strain relationship for Gilmore, 1% cement treatment, 7-day cure.

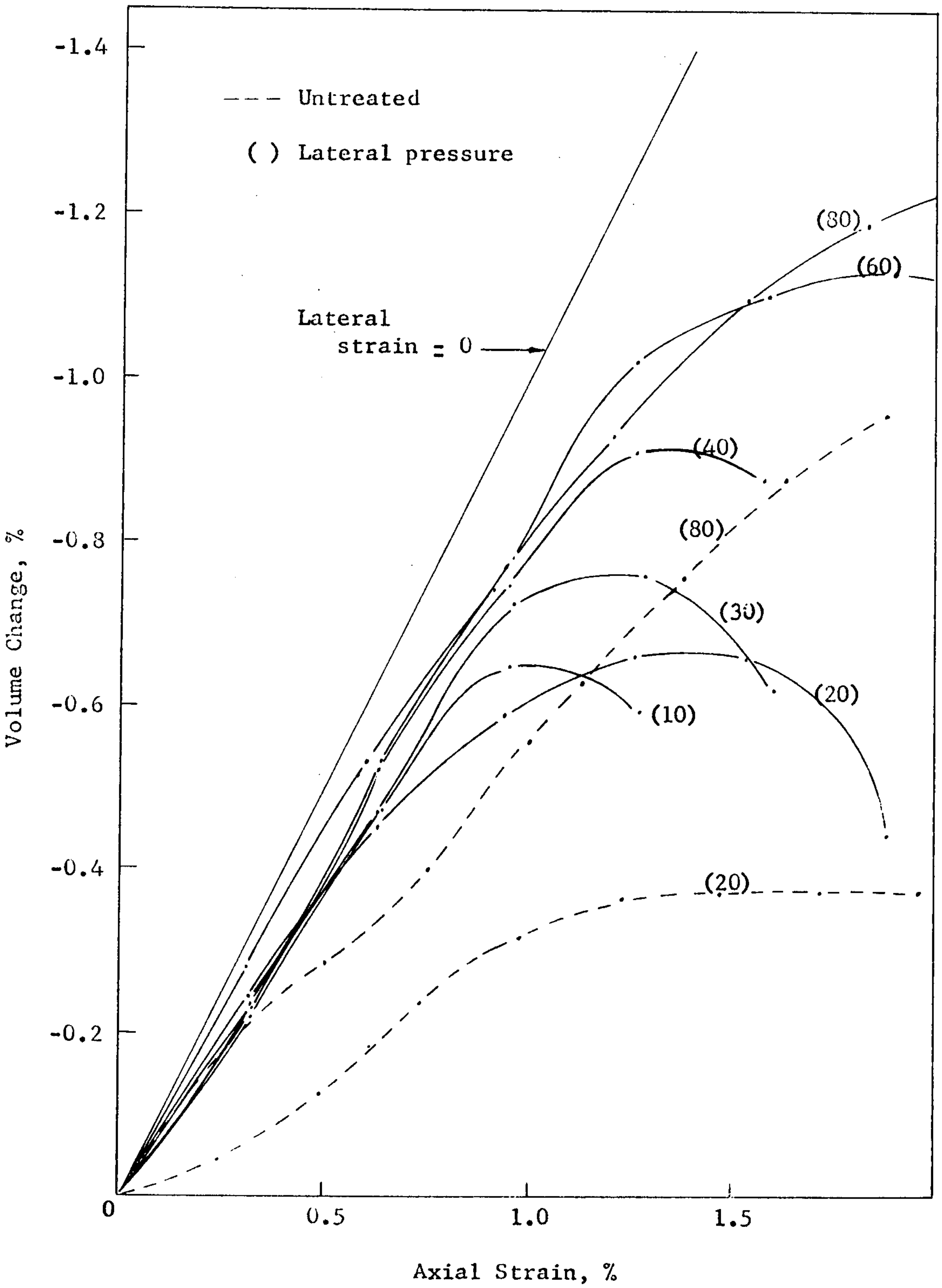

Figure 34. Volume change-axial strain relationship for Gilmore, 3% cement treatment, 7-day cure

indicating that the material is undergoing a limited amount of lateral strain even though the volume is decreasing. The slope of the volume change-strain curves for the cement treated materials is much closer to the condition of Poisson's ratio equal to zero, which can occur only when lateral strain is very small. Using the previous assumption that lateral strain tends to increase lateral support, the cement treated materials have very little tendency to increase lateral support prior to the point of minimum volume due to the small amount of lateral strain developed. The effective stresses at the point of minimum volume should therefore be closely related to shear strength occurring under field conditions.

The untreated materials may tend to develop lateral strain even during light loadings, resulting in some increase in lateral support before the condition of minimum volume is reached. Thus the effective stresses at the point of minimum volume change, as determined under conditions of constant lateral pressure, may not be achieved under field conditions, but at least may be closer indications of potential field strength than lab strengths at maximum effective stress ratio.

Strength of cement treated crushed stone prior to minimum volume is primarily a function of the mixture. Strength of the untreated material under field conditions appears to be more closely related to the ability to develop lateral support than the strength characteristics of the material itself.

Shrinkage cracking, that develops as the cement treated rolled stone base cures, could be detrimental to the strength of the base due to a reduction of lateral support in the region of any cracking. If the amount of shrinkage is excessive, a large amount of lateral deflection would be required to build up lateral support which can only occur after the ultimate strength of the material is exceeded and the cement bonds begin to rupture. This process could occur adjacent to cracks in the base course and though it increases the amount of lateral support, the shear strength might actually be reduced. The smaller the quantity of cement added, however, the less the magnitude of cracking of cement treated crushed stone bases. While shrinkage studies were not conducted as a part of this research, it is generally thought that up to 3% cement by dry weight would not result in excessive cracking[1], though maintaining a much higher degree of total stability than the untreated stone.

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[1]Further studies are needed to substantiate this hypothesis, although axial expansion measurements of freeze-thaw test specimens of the cement-treated stones by Merrill and Hoover tend to support the generality.

SUMMARY

The objective of this investigation was to observe and analyze the effects of type I Portland cement on the stability of three crushed limestones.

Specimens of the crushed stones containing 0, 1, and 3% by dry weight of portland cement, cured for periods of 7 and 28 days, were tested by consolidated-undrained triaxial shear methods including measurement of pore water pressures and change of volume.

Shear strength parameters of cohesion, c, and angle of internal friction, ϕ, were determined on the basis of two failure criterion, i.e., maximum effective stress ratio and minimum volume. As previously indicated (12), the magnitude of the difference in values of shear strength at the two criteria of failure may be an indication of the amount of interlocking within a granular material. Shear strength based on the failure criteria of maximum effective stress ratio is normally the greater due to the interlocking of particles and generally results in increased cohesion coupled with slight decrease in friction angle. All untreated materials in this investigation analyzed by the two criteria of failure followed the above pattern. The addition of cement in the Bedford and Gilmore stones resulted in similar shifting of shear parameters when analyzed by the two failure criteria but were of greater magnitude than those of the untreated. The Garner stone treated with 3% cement followed a similar pattern, whereas the 1% cement treatment increased both angle of friction and cohesion. Thus, in general, the addition of cement in the three crushed stones resulted in an increase in the shear

strength parameter cohesion, and was possibly due to an increased condition of interlocking.

As previously noted (4), cement treatment of granular materials results in a relatively constant angle of friction, whereas cohesion increases rapidly with increased cement content. However, addition of cement to the three crushed stones in this investigation produced varying values of shear strength parameters with increasing cement contents. Cohesion of the treated Bedford stone increased by as much as 72 psi, while the angle of friction remained relatively constant, as compared with the zero percent cement specimens. Cohesion of the treated Garner stone increased nearly linearly with increase in cement content after 28 days cure. However, ϕ reduced slightly at 1% cement then increased at 3% cement content. Addition of 1% cement to the Gilmore stone produced relatively no changes in cohesion but increased ϕ by about six degrees above that of the untreated Gilmore. At 28 days cure, the addition of 3% cement in the Gilmore produced no additional change in ϕ but significantly increased cohesion. It is felt that addition of 1% cement in the Gilmore may not result in a complete cementation, or bonding of the large aggregate, but rather in a bonding of the fines, increasing the interlocking frictional effects between the stabilized fines and the larger aggregates.

Addition of cement to the three crushed stones reduced pore pressures to insignificant quantities. Change of pore pressure from failure conditions of minimum volume to maximum effective stress ratio may indicate the magnitude of expansion occurring during this phase of shear. Treatment of the crushed stones with cement significantly reduced the magnitude of the above change and was most pronounced with the Bedford stone, possibly due to reduction of plasticity of the fines.

Cement treatment reduced the quantity of strain required to achieve ultimate strength by either criteria of failure as compared with the untreated materials. Magnitude of strain at failure for all three treated stones was relatively independent of lateral, or confining, pressures but appeared to vary with cement content and length of cure, i.e., decreased with increasing cement content and cure period. Magnitude of strain at failure of the untreated stones generally increased with increasing lateral pressures.

Analysis of volume change characteristics of the cement treated materials led to the premise that shear strength alone does not fully explain the behavior of a granular material under actual field conditions. As the untreated materials were axially loaded, there may have occurred a reduction in volume as well as a small quantity of lateral strain. In a base course, tendency for lateral expansion may be resisted by the adjacent material resulting in increased lateral support. This suggests that stability of a granular material is not entirely a function of the shear strength but must also be a function of the lateral restraining support that can be developed and the amount of expansion required to achieve this support.

The addition of cement to the three granular materials reduced the amount of lateral strain developed up to the point of minimum volume failure criteria, resulting in a potential Poisson's ratio of near zero. Thus the strength properties of the cement treated materials at the point of minimum volume may more adequately represent field strength and stability conditions, than use of the strength properties at maximum effective stress ratios.

CONCLUSIONS

1. Ultimate strength of a treated granular base course material may not be the main criterion for use in highway design.
2. Stress conditions at the point of minimum volume may be more closely related to actual field conditions than maximum effective stress ratio, due to a decrease of magnitude of lateral strain.
3. Mechanism of stabilization resulting from the addition of cement is not uniform for the crushed stone materials used in this investigation. Addition of cement to the Bedford stone appears to increase the cohesion of the material with little effect on the frictional parameter. Addition of 1% cement to the Garner stone appears to affect both cohesion and the angle of internal friction, while the addition of 3% cement results in a large increase in cohesion with little change in the angle of internal friction from that of the untreated material. Addition of 1% cement to the Gilmore stone has a marked effect on the friction parameter but little on the cohesion; additional cement has no further effect on the friction parameter but tends to increase cohesion.
4. Cement treatment significantly reduces pore pressures developed in all three crushed stones during shear and may indicate a general reduction in the overall compressibility of the material.
5. Amount of strain at failure decreases with increased cement content or length of cure.

6. Stability of an untreated granular base course may be dependent upon the amount of lateral restraint that exists prior to loading and the ability to increase this restraint through resistance to lateral expansion within the loaded area. Addition of cement reduces the amount of lateral expansion developed prior to the failure condition of minimum volume. Thus the stability of the cement treated material prior to minimum volume appears to be a function of the material rather than the conditions of increasing lateral support found with the untreated material.

Further Investigations

The authors feel there is a need for further research into the lateral deformation characteristics of granular materials through direct measurement of lateral stress and strain. Of particular importance are the deformation characteristics of dynamic, rather than relatively static conditions of axial loading. Eventually, quantitative field tests should be conducted to determine the magnitude and manner of development of lateral restraint. Thus knowing the reaction of the material under triaxial static and dynamic loadings, and the manner in which the material resists lateral deformations under field conditions, a method of test could be developed wherein actual conditions of increasing lateral support occurring under field conditions might be simulated in the laboratory.

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APPENDIX

Table 7. Summary of triaxial test data for Bedford specimens with 1% cement additive

Lateral pressure psi	Initial		Minimum volume condition				Maximum effective stress ratio			
	Moisture content,%	Dry density, pcf	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume change,%	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume change,%
<u>7-day cure</u>:										
10	10.56	127.8	164.6	9.3	1.99	-0.78	190.7	9.5	2.62	-0.77
20	9.63	124.2	254.7	19.8	1.71	-0.69	268.8	20.1	2.34	-0.64
20	9.81	127.0	234.3	19.7	1.26	-0.68	282.4	20.6	2.21	-0.12
30	9.59	121.3	267.2	29.7	1.52	-0.74	296.7	29.9	2.74	-0.28
30	9.04	125.8	283.8	29.5	1.41	-0.57	325.0	30.1	2.35	-0.29
40	9.45	122.0	311.5	39.4	1.43	-0.69	351.6	39.8	2.66	-0.37
40	9.67	123.9	292.2	39.2	1.54	-1.11	348.3	39.5	3.07	-0.73
60	9.79	126.4	482.9	59.3	2.40	-1.03	516.1	59.4	3.69	-0.67
80	10.40	126.3	638.5	78.4	3.54	-1.23	640.4	78.4	3.86	-1.17
<u>28-day cure</u>:										
10	10.86	125.7	132.7	9.5	1.60	-0.64	226.6	10.9	3.85	-0.07
20	9.97	126.9	307.4	19.8	1.69	-0.87	334.2	20.6	2.32	-0.43
30	9.42	126.3	378.6	29.7	1.58	-1.02	410.9	29.7	2.21	-0.62
30	9.83	125.5	320.5	29.2	1.54	-1.01	369.5	29.3	2.46	-0.72
40	9.66	126.1	411.3	39.7	2.06	-1.34	443.4	40.6	3.00	-0.94
40	10.03	126.5	384.3	39.6	1.89	-1.09	442.4	39.8	2.83	-0.76
60	9.71	123.4	525.3	58.9	2.91	-1.69	526.0	59.0	3.22	-1.67
60	9.66	126.9	432.6	59.3	1.58	-1.00	523.1	59.1	3.78	-0.48
80	9.84	125.5	608.5	78.9	2.21	-1.31	647.2	78.9	3.15	-1.05
80	10.09	127.1	588.4	78.6	2.23	-1.42	648.8	78.3	3.48	-1.28

Table 8. Summary of triaxial test data for Bedford specimens with 3% cement additive

Lateral pressure psi	Initial		Minimum volume condition				Maximum effective stress ratio			
	Moisture content,%	Dry density, pcf	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume change,%	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume change,%
7-day cure:										
10	9.93	127.6	369.1	9.5	0.97	-0.61	424.0	10.2	1.29	-0.37
20	10.12	126.8	438.0	20.0	1.07	-0.81	482.2	20.6	1.38	-0.64
30	10.12	126.9	480.4	30.4	1.18	-0.83	523.1	30.4	1.82	-0.43
40	10.73	125.7	592.4	38.5	1.50	-1.16	577.1	38.8	2.14	-0.84
60	10.38	126.5	650.9	59.2	1.60	-1.14	703.8	59.4	2.23	-0.91
80	10.06	128.4	850.8	79.7	2.08	-1.39	876.1	79.8	2.40	-1.28
28-day cure:										
10	10.00	123.6	422.4	10.0	0.91	-0.66	454.1	10.6	1.21	-0.59
20	10.19	122.7	492.9	19.9	1.05	-1.06	520.7	20.6	1.36	-0.91
30	10.13	121.6	474.6	29.7	0.92	-1.00	512.4	30.0	1.23	-0.93
40	9.18	122.8	595.3	39.7	1.03	-0.92	633.8	40.0	1.34	-0.82
60	9.18	124.8	257.1	59.8	1.75	-1.17	774.7	60.2	2.07	-1.16
80	11.41	123.2	805.5	78.8	2.53	-1.49	822.0	78.8	3.16	-1.40

Table 9. Summary of triaxial test data for Garner specimens with 1% cement additive

Lateral pressure psi	Moisture content,%	Dry density,pcf	Minimum volume condition $\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume change,%	Maximum effective stress ratio $\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume change,%
<u>7-day cure</u>:										
10	7.68	139.5	143.6	10.0	0.95	-0.52	244.0	11.3	1.90	+0.10
20	6.70	140.4	266.9	19.7	0.95	-0.66	362.8	21.0	1.90	-0.03
30	7.85	141.6	280.4	29.6	0.85	-0.63	454.3	31.1	2.44	-0.13
40	6.35	142.3	392.7	39.5	1.19	-0.82	525.1	39.9	2.47	-0.19
60	6.26	144.7	627.7	58.8	1.91	-1.09	755.4	59.2	2.92	-0.71
80	6.95	142.3	677.7	79.3	1.67	-1.04	820.3	79.8	2.95	-0.69
<u>28-day cure</u>:										
10	6.94	140.0	222.3	10.0	1.29	-0.57	279.9	11.1	1.94	+0.07
20	7.38	139.5	249.4	19.5	0.79	-0.52	385.1	21.4	2.08	+0.13
30	6.60	141.8	352.8	29.3	1.31	-0.80	484.6	29.7	2.62	-0.08
40	6.75	142.4	365.1	39.4	1.14	-0.89	528.6	40.6	2.43	-0.34
60	6.26	140.0	545.4	59.8	1.56	-0.89	650.5	60.7	2.81	-0.44
80	7.01	135.3	633.9	79.6	1.50	-0.99	670.6	79.9	2.11	-0.85

Table 10. Summary of triaxial test data for Garner Specimens with 3% cement additive

Lateral pressure psi	Initial		Minimum volume condition				Maximum effective stress ratio			
	Moisture content,%	Dry density,pcf	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume change,%	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume change,%
7-day cure:										
10	5.89	139.5	445.1	9.7	0.14	-0.60	647.1	10.1	0.46	-0.51
20	6.00	136.2	652.6	20.0	1.58	-0.87	652.6	20.0	1.58	-0.87
20	5.98	138.7	703.2	20.0	0.65	-0.68	761.2	20.5	0.91	-0.49
30	6.14	135.5	598.4	29.8	0.69	-0.73	672.5	30.1	1.00	+0.01
30	5.16	137.7	613.0	29.8	0.63	-0.81	709.6	30.1	0.94	-0.78
40	5.71	138.4	657.6	39.9	0.96	-0.83	748.3	39.9	1.28	+1.28
60	5.55	139.2	873.5	59.9	1.09	-1.00	918.1	60.4	1.43	-0.90
80	5.84	137.7	1053.9	79.7	0.71	-0.62	1184.4	80.3	1.35	-0.29
28-day cure:										
10	6.56	136.9	617.8	9.8	0.95	-0.60	623.3	10.5	1.25	-0.31
20	5.84	137.7	689.3	19.6	0.67	-0.85	787.3	19.7	0.98	-0.78
30	5.89	136.6	727.3	29.9	0.67	-0.48	727.3	29.9	0.67	-0.48
40	6.08	138.9	832.0	39.2	1.10	-0.39	885.6	39.3	1.42	-0.25
60	6.12	140.4	1142.9	59.7	1.27	-0.51	1163.2	59.7	1.58	-0.29
80	5.95	137.3	1183.8	79.2	0.95	-0.69	1206.3	79.2	1.26	-0.60

Table 11. Summary of triaxial test data for Gilmore specimens with 1% cement additive

Lateral pressure psi	Initial: Moisture content, %	Initial: Dry density, pcf	Minimum volume condition: $\bar{\sigma}_1$ psi	Minimum volume condition: $\bar{\sigma}_3$ psi	Minimum volume condition: Strain %	Minimum volume condition: Volume change, %	Maximum effective stress ratio: $\bar{\sigma}_1$ psi	Maximum effective stress ratio: $\bar{\sigma}_3$ psi	Maximum effective stress ratio: Strain, %	Maximum effective stress ratio: Volume change, %
7-day cure:										
10	7.33	131.1	84.4	9.0	1.27	-0.52	189.1	10.5	3.18	+0.56
20	7.35	128.5	164.8	19.4	1.23	-0.77	245.3	20.5	3.38	+0.14
30	7.62	132.6	238.2	19.0	1.81	-0.81	355.1	30.6	4.23	-0.19
40	7.62	132.4	329.2	38.7	1.72	-0.77	409.3	39.4	3.66	-0.12
60	7.80	134.7	484.1	57.3	2.53	-1.26	560.2	59.0	4.58	-0.51
80	7.55	131.4	578.0	78.0	2.97	-1.11	618.8	78.6	4.20	-0.97
28-day cure:										
10	7.61	130.9	127.2	9.4	1.26	-0.63	106.4	10.4	1.53	-0.19
10	7.53	130.7	110.2	9.6	1.06	-0.57	195.4	10.8	2.65	+0.35
20	7.11	128.8	184.1	19.1	1.35	-0.79	250.2	19.6	2.59	-0.44
30	7.52	129.7	227.6	29.2	1.09	-0.71	335.5	30.0	2.91	+0.31
40	7.77	129.7	332.6	38.8	1.94	-0.95	404.4	39.8	3.87	-0.37
40	7.26	129.6	304.8	37.8	1.91	-1.06	386.6	38.7	3.82	-0.73
60	7.56	135.4	500.6	58.2	2.03	-0.95	596.7	59.2	3.68	-0.39
80	7.95	134.0	650.7	77.0	3.13	-1.66	694.1	77.8	4.42	-1.45

Table 12. Summary of triaxial test data for Gilmore specimens with 3% cement additive

Lateral pressure psi	Initial		Minimum volume condition				Maximum effective stress ratio			
	Moisture content, %	Dry density, pcf	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain %	Volume change, %	$\bar{\sigma}_1$ psi	$\bar{\sigma}_3$ psi	Strain, %	Volume change, %
7-day cure:										
10	7.11	130.6	289.9	9.7	0.65	-0.65	341.9	10.7	1.28	-0.25
20	7.35	131.8	380.1	19.2	1.06	-0.66	445.0	19.4	1.69	-0.45
30	7.06	134.0	453.9	29.6	1.29	-0.76	549.4	30.3	2.25	-0.25
40	6.68	136.6	520.2	39.3	1.26	-0.91	633.4	40.2	2.20	-0.47
60	6.97	131.8	651.7	59.2	1.79	-1.13	687.5	59.8	2.41	-0.81
80	7.17	133.6	796.7	78.9	1.77	-1.25	848.1	79.0	2.68	-1.13
28-day cure:										
10	6.66	135.3	423.2	9.3	0.72	-0.97	483.6	9.7	1.01	-0.90
20	6.93	131.9	487.1	19.0	0.64	-0.89	536.1	19.7	0.95	-0.72
20	6.34	127.8	577.3	19.9	1.02	-0.92	607.5	10.0	1.34	-0.69
30	7.25	128.0	432.9	29.7	0.90	-0.62	512.8	30.2	1.80	-0.30
40	6.74	128.8	538.8	39.8	1.48	-0.99	575.8	40.2	2.08	-0.68
40	7.46	130.3	628.5	39.6	1.08	-0.77	703.1	40.2	1.72	-0.55
60	6.87	132.1	745.4	59.1	1.77	-1.27	786.8	59.2	2.31	-0.93
80	7.29	135.4	960.0	78.6	2.01	-1.45	1008.6	78.7	2.68	-1.10