Deformation Moduli of Asphalt Cement-Treated Granular Materials

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Abstract. Triaxial compression studies of asphalt cement-treated granular materials indicate that each material exhibited a volume decrease during initial phases of stress-strain. The stress-strain curve was nearly linear until the material reached minimum volume, after which curvature increased rapidly. The analysis presented is based on the assumption that the point of minimum volume represents a yield point similar to proportional limit. Deformation moduli, M, were evaluated by applying linear regression analyses to that portion of the effective stress ratio versus strain curves occurring prior to minimum volume, instead of using a tangent or secant modulus. Plots of M versus σ_3 were analyzed to determine the relationship between M and the confining pressure, σ_3. Poisson's ratio, μ, was evaluated by assuming that volume change is a function of either octahedral or deviator stresses.

Results of linear regression analyes showed that deformation moduli will generally tend to increase with increasing lateral constraint. However, statistical analyses of M versus σ_3 plots indicated that this relationship is not well defined and in many cases M is essentially constant for the range of lateral pressures studied. By using average values of M, it was found that limestone and dolomitic aggregates treated with asphalt cement have moduli generally greater than gravel. Both method of evaluation of μ gave values generally in the range of 0.36 — 0.44, indicating that volume change was primarily a function of deviator stress.

Any soil or granular material used in the design and construction of a flexible pavement must ultimately satisfy two related criteria: (1) adequate shear strength, and (2) resistance to excessive settlement or deformation that impairs trafficability. Thickness design of base courses for flexible pavements requires a knowledge of these relationships.

The study presented herein is an analysis of triaxial compression tests performed on asphalt cement-treated granular base course materials used in the construction of several Iowa highways. A linear model is hypothesized for deformation occurring prior to and including minimum volume. Linear regression analyses of stress-strain behavior were employed to develop moduli which show a relationship between shear strength and deformation. These analyses also led to an evaluation of Poisson's ratio for the materials utilized in the study.

Literature Review

A common method of expressing shear strength of a soil is Coulomb's equation

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$$S = C + N \tan \delta, \quad (1)$$

where: N = pressure normal to plane of failure,
C = cohesion, and
δ = angle of internal friction.

Shear strength of untreated granular materials is primarily a function of internal friction; when treated with stabilizing agents, they have been shown to develop shear strength as a function of both cohesion and internal friction.

In the late 1940's, McLeod[1] made an extensive study of the c and δ properties of bituminous treated materials in order to achieve a logical basis for designing flexible pavements. It was determined that the stability of an asphalt-treated material is a function of thickness, lateral support, friction between pavement and tires, frictional resistance between base and pavement, pressure distribution, and the rate and nature of loading. McLeod asserted that the primary sources of stability are cohesion, internal friction, and viscous resistance.

Marshall[2] developed a method of measuring strength and stability of granular materials treated with penetration grades of asphalt cement. Properties studied in the Marshall method are bulk specific gravity, stability and flow, density, and percent air voids. Two other well-known design methods for asphalt-treated mixes are termed the Hveem and Hubbard-Field. All three methods are widely used and are currently recommended by the Asphalt Institute[2].

Due to the anisotropic nature of soil, direct applications of the theory of elasticity are greatly hampered. A material is said to behave elastically if the strain created during application of a stress disappears when that stress is removed. If all or part of the strain remains after removal of a stress, there is a residual inelastic deformation often termed permanent set or plastic deformation. If stress and strain are proportional, the material is linearly elastic since such behavior satisfies Hooke's law. The slope of a stress-strain curve is

$$E = \frac{\sigma}{\epsilon}, \quad (2)$$

where: E = modulus of elasticity,
σ = stress, and
ϵ = strain.

This relationship is illustrated in Fig. 1a.

Many materials, especially metals, exhibit linear behavior at lower stress levels. The point at which the stress-strain curve begins to deviate from linear behavior is called the proportional limit. If stress levels are beyond the proportional limit or the material has a nonlinear stress-strain curve, the tangent modulus and secant modulus illustrated in Fig. 1b are often used as expressions of approx-

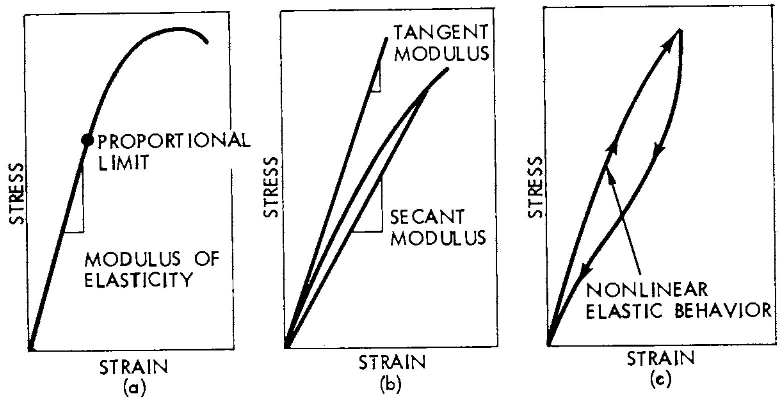

Fig. 1. Typical stress-strain curves.

imate moduli of elasticity. On the other hand, some materials behave elasticially, but not linearly, as illustrated in Fig. 1c.

A second important elastic constant is Poisson's ratio,

$$\mu = \frac{\epsilon_1}{\epsilon_3}, \tag{3}$$

or the ratio of lateral to longitudinal strain. In his memoirs, Poisson[3] wrote that μ would be 0.25 for pure elastic behavior.

For common metals, notably steel, μ is about one-third. For dense soils, concrete, and sandstone, Terzaghi[4] reported that μ may vary from about 0.2 at low stress levels to 0.5 at high stress levels. Rubber is an example of a material which has little or no volume change when stressed and therefore has a value of μ approaching 0.5. Cork has little or no lateral strain when stressed though the volume is reduced, and has a value of μ approaching 0.0.

Taylor[5] stated that the meaning of μ is not easy to define for soil media and suggested the use of the volumetric stress-strain relationship, illustrated in Fig. 2a. Terzaghi[4] discussed a similar technique for observing deviation from elastic behavior for soils as illustrated in Fig. 2b. The term $\Delta V_0/V$ is volume change caused by original confining pressure in a triaxial compression test, while $\Delta V/V$ is volume change occurring during shear phase of the test. Deviation from linear-elastic behavior can be observed from these types of plots.

Moore[6] studied the variations of Poisson's ratio from triaxial soil tests, solving the equations of elasticity for ideal conditions. He illustrated differences between the real value of Poisson's ratio and the apparent value obtained from triaxial test data and developed correction factors applicable to the apparent values of μ and E. For real values of Poisson's ratio greater than about 0.2, the value of the apparent Poisson's ratio was slightly gerater and the true

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values of E were about 7% greater than apparent values of E.

Behavior of an asphalt-treated material is partly a function of the viscosity of the asphalt. An important characteristic of viscous flow is continued deformation at a decreasing rate when a constant stress is applied. On the other hand, if a constant deformation is imposed on a viscous material, there will be a decrease in the stress at a decreasing rate with increasing time[7,8].

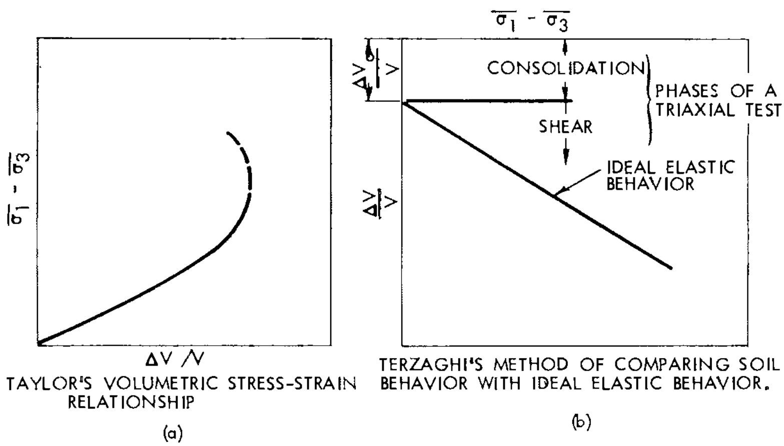

Fig. 2. Typical stress-volume change relationships.

In an effort to determine if sand-asphalt mixtures exhibit linear viscoelastic behavior, Huang[9] studied the possibility of such materials satisfying the principles of superposition for an isotropic medium. He found that these materials did not exhibit linear viscoelastic behavior for the entire duration of a triaxial test and anticipated that the theory of linear viscoelasticity applied only in the strain range of less than 1%. He also suggested that it may be reasonable to assume that asphalt mixtures are nearly incompressible, i.e., Poisson's ratio is about 0.5.

In a study of the anisotropic behavior of bituminous mixes, Busching, Goetz, and Harr considered characterizing deformation as a total differential equation involving such parameters as time, temperature, stress, and asphalt content[10]. They concluded that Poisson's ratios determined from volume change measurements did not agree with those determined from linear measurements and showed that Poisson's ratio tends to increase with time, ranging from 0.43 to 0.52 during a test. For short periods of time, μ could probably be considered as a constant.

Didukh and Ioselevich[11] summarized four possible theories for characterizing soil deformations: (1) elasticity; (2) small elastoplastic deformations; (3) small plastic deformations of a dispersed soil system; and (4) the theory of "soil dynamics." In each case, they emphasized that only small deformations could be considered if any of the theories were to be applicable.

Method of Test

Materials

Eight materials, representing seven aggregate types and sources were used in this study. Table 1 presents a limited number of the engineering properties of the materials tested. The field mixed samples were obtained by Iowa Highway Commission (IHC) personnel from construction batch plants immediately following mixing with asphalt.

Table 1. Engineering properties of the materials tested.

Material designation	Aggregate type	Aggregate specific gravity	Asphalt cement content[a], %	Mixing conditions	Marshall density[b], pcf
1734	Limestone	2.65	4.27	Field	138
			4.06	Lab	—
			4.97	Lab	—
1846	Limestone	2.76	3.80	Field	143
			4.05	Lab	—
			4.98	Lab	—
1822	Dolomite	2.76	4.07	Field	142
			3.93	Lab	—
			4.87	Lab	—
1855	Dolomite with chert	2.75	4.13	Field	142
			4.30	Lab	—
728	Limestone and dolomite	2.70	4.47	Field	142
			4.49	Lab	—
1903	Gravel	2.72	5.33	Field	149
			5.06	Lab	—
1904	Gravel	2.72	3.80	Field	145
			4.09	Lab	—
1485	Gravel (sand)	2.19	4.13	Field	136
			3.93	Lab	—

[a] Asphalt contents of field mixes are the extracted values furnished by the Iowa State Highway Commission. Asphalt contents of the lab mixes are based on the dry weight of the aggregate.

[b] Marshall densities are based on the external volume of the specimen minus the volume of the voids.

Aggregate for the lab-mixed samples was also procured by IHC

FIELD MIXES

LABORATORY MIXES

SIEVE ANALYSIS OF AGGREGATES

COMBINING AND MIXING OF AGGREGATES

SIEVE ANALYSIS OF AGGREGATE MIXTURES

HEATING OF AGGREGATE AND ASPHALT TO 250 — 260°F

MIXING OF AGGREGATES AND ASPHALT

HEATING OF SPECIMEN SIZE PORTIONS OF MIXES UNTIL 250 — 260°F TEMPERATURE WAS MAINTAINED.

MOLDING OF TEST SPECIMENTS IN HEATED MOLD

WEIGHING AND MEASURING OF SPECIMENS

CURING OF SPECIMENS

WEIGHING AND MEASURING PRIOR TO TRIAXIAL TEST

Fig. 3. Specimen preparation flow chart.

personnel by sampling the aggregate just prior to introduction into the batch plant or from stockpiled materials. Asphalt cement was added and mixed in the laboratory. An asphalt cement of 120 — 150 penetration grade was utilized for both field and lab mixes.

Specimen Preparation

Test specimens used in this study were prepared in accordance with the procedure outlined in Fig. 3. Four-in.-diameter by 8-in. high cylindrical specimens were molded by a vibratory compaction procedure previously reported by Hoover[12]. This procedure results in densities equivalent to those obtained by the Marshall method, while reducing the quantity of aggregate degradation and segregation normally appearing under drop hammer techniques.

Testing Procedure

All tests were performed in the double-bay triaxial compression machine shown in Fig. 4. Designed by the ERI Soil Research Laboratory, and built by the ERI Fabrication Shop, the equipment is capable of (1) testing two samples simultaneously, (2) deformation rates varying from 0.0001 to 0.1 in. per minute, (3) volume change measurements to less than 0.01 cubic in., (4) maximum axial loading capacity of 11,000 pounds per cell, and (5) lateral pressure capacities of 0 to 100 psi.

The testing procedure followed during this investigation is noted in Fig. 5. For each specimen, pore pressure, volume change, and

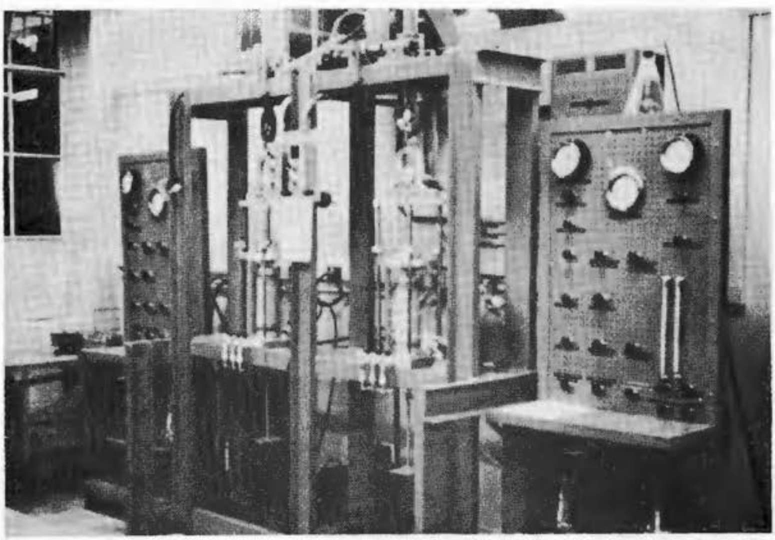

Fig. 4. Double-bay triaxial compression testing machine.

axial load data were recorded at vertical deflection intervals of 0.01 in. for the first 0.2 in. of deformation and at intervals of 0.025 in. during the remainder of the test. A Fortran IV program, developed for the IBM 360/65 computer was used to calculate stress, strain, volume change, and pore pressure for each data point in a test. A Calcomp Digital Incremental Plotter was utilized for continuous plots of effective stress ratio, pore pressure, and volume change versus longitudinal strain for each test specimen.

SPECIMEN SATURATED UNDERWATER IN VACUUM JAR BY APPLYING FULL VACUUM FOR MINIMUM OF 45 MINUTES.

SPECIMEN PREHEATED UNDERWATER TO 100°F

SPECIMEN PLACED IN TRIAXIAL CELL.

CELL FILLED WITH DE-AIRED WATER. CELL HEATER TURNED ON AND 100°F. TEMPERATURE MAINTAINED FOR AT EAST 30 MINUTES PRIOR TO, AND DURING TESTING.

SPECIMEN CONSOLIDATED UNDER σ_3 LATERAL PRESSURE FOR MINIMUM OF 36 MINUTES. VOLUME CHANGE DATA RECORDED.

SPECIMEN SHEARED. VOLUME CHANGE, AXIAL LOAD, DEFORMATION, AND PORE PRESSURE DATA RECORDED.

TRIAXIAL TEST COMPLETE.

Fig. 5. Specimen test procedure flow chart.

Method of Analysis

Linear elastic theory is based upon the hypothesis that a proportional relationship exists between stress and strain. If the material being considered is linearly elastic, loading and unloading curves are assumed to coincide unless the proportional limit is exceeded. In soils, loading and unloading curves usually do not coincide. However, it has been observed that at lower stress levels, some soils will exhibit stress-strain behaviors that may be approximated by equations of elasticity. Taylor[5] has stated that it is not important that a material behaves elastically, but if a constant stress-strain ratio can be established, the equations of elasticity may be applicable.

Theory shows that an elastic material will exhibit a volume increase when in tension and a volume decrease when under compression. Since most soils have little or no tensile strength, and because compressive loading stresses are normally encountered in soils, the equations of elasticity applied to soil behavior will probably be most applicable during that phase of deformation in which volume decrease occurs. The deformation model for asphalt cement-treated granular materials developed herein is therefore based upon the assumptions that (1) the stress-strain curve is approximately linear prior to and including the point of minimum volume in a triaxial compression test, and (2) the equations of elasticity will adequately describe this portion of the deformation of such materials. Thus the ratio of stress to strain is herein referred to as a deformation modulus, M, rather than a modulus of elasticity.

Triaxial compression specimens are usually consolidated to equilibrium under a given confining pressure prior to beginning the shear phase of the test, as shown in Fig. 6. When the shear phase of the test is begun and the vertical stress, σ_1, is applied, the stress which acts to deform the specimen is the difference between the verticial stress and the original confining pressure, $\sigma_1 - \sigma_3$, often referred to as the deviator stress, D*. The deformation modulus may therefore be defined as

$$M = \frac{D}{\epsilon_1}, \tag{4}$$

where: ϵ_1 = longitudinal strain.

An illustration of typical stress-strain-volume change pore-pressure behavior observed for the materials in this investigation is shown in Fig. 7. The effective stress ratio, ESR, is

$$ESR = \frac{(\overline{\sigma_1} - \overline{\sigma_3})}{\overline{\sigma_3}}, \tag{5}$$

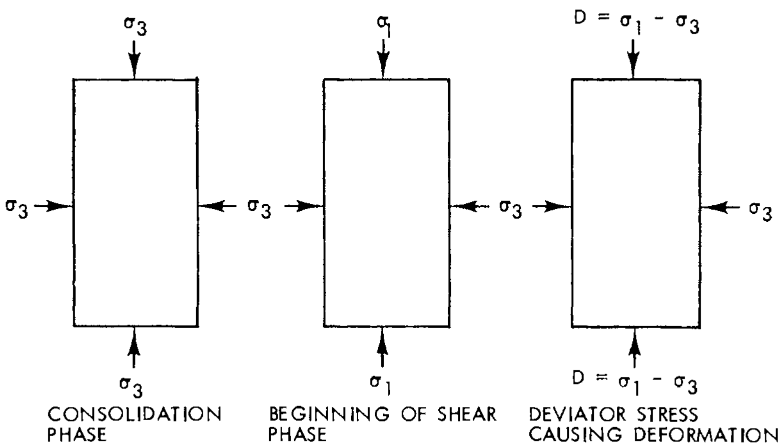

Fig. 6. Change of stress conditions in a triaxial test.

*In effective stress terms, $D = \overline{\sigma_1} - \overline{\sigma_3}$.

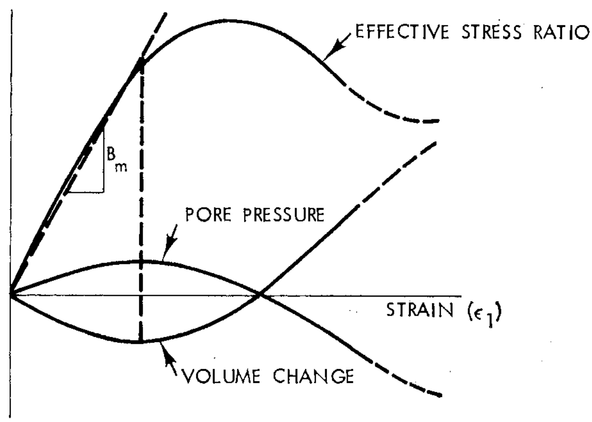

Fig. 7. Representative stress-strain-volume change-pore pressure behavior of asphalt cement-treated granular materials.

where: $\bar{\sigma}_1 = \sigma_1 - u$,

$\bar{\sigma}_3 = \sigma_3 - u$, and

u = pore pressure.

Unit volume change is defined as the ratio of the change in volume to the original volume, $\Delta V/V$. When unit volume change is negative, pore pressure is positive, and when volume change becomes positive, pore pressure becomes negative, as shown in Fig. 7.

If the slope of the ESR curve prior to the point of minimum volume is assumed linear and is denoted as B_m, then

$$B_m = \frac{ESR}{\epsilon_1}. \tag{6}$$

Multiplying each side of Eq. (6) by $\bar{\sigma}_3$ leads to

$$\bar{\sigma}_3 B_m = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{\epsilon_1} = \frac{D}{\epsilon_1} = M, \tag{7}$$

a means of evaluating deformation moduli.

Poisson's ratio must be essentially constant if the equations of elasticity are to be valid for describing the initial deformation of asphalt cement-treated granular materials. One method of testing the validity of the equations of elasticity to soil behavior is to use the approach of Terzaghi[4] and assume that the relationships of Eqs. (2) and (3) are true (also see Fig. 2b). If a triaxial specimen is allowed to consolidate to equilibrium under a confining pressure $P = \sigma_{1,2,3}$, strain in the three principal planes is

$$\epsilon_{1,2,3} = \frac{P}{M}(1 - 2\mu), \tag{8}$$

where the modulus of elasticity has been replaced by the modulus of deformation, M. Summation of the three principal strains during consolidation is the unit volume change

$$\frac{\Delta V_0}{V} = \epsilon_1 + \epsilon_2 + \epsilon_3 \,, \tag{9}$$

or

$$\frac{\Delta V_0}{V} = \frac{3P}{M}(1 - 2\mu). \tag{10}$$

As σ_1 is increased during the shear phase of the triaxial test (Fig. 2b) and σ_3 is held constant and equal to σ_2, there will be an additional unit volume change of

$$\frac{\Delta V}{V} = \frac{1-2\mu}{M}(\sigma_1 + 2\sigma_3). \tag{11}$$

If the quantity $(\sigma_1 + 2\sigma_3)$ is divided by 3, Eq. (12) becomes

$$\frac{\Delta V}{V} = \frac{1-2\mu}{M} \times 3\sigma_0 \,, \tag{12}$$

where σ_0 is the octahedral stress acting upon the material. Poisson's ratio at any point in a triaxial test may then be determined by manipulating Eq. (12) into

$$\mu = \frac{1}{2} - \frac{(\Delta V/V) \times M}{6\sigma_0} \,. \tag{13}$$

A more general approach of evaluating μ, involving both consolidation and shear phases of a triaxial test, is to assume that unit volume change is caused by the deviator stress, D. Figure 2b shows that volume change in a triaxial test is

$$\frac{\Delta V}{V} = \frac{\Delta V_0}{V} + B_v D \,, \tag{14}$$

where B_v is the slope of the line. If $3\sigma_0 = (\sigma_1 + 2\sigma_3)$ and $D = (\sigma_1 - \sigma_3)$, then

$$3\sigma_0 = D + 3\sigma_3 \,. \tag{15}$$

Substitution into Eq. (12) leads to

$$\frac{\Delta V}{V} = \frac{1-2\mu}{M}(D + 3\sigma_3). \tag{16}$$

The only factor causing a unit volume change during the shear phase is the deviator stress, D. If $\Delta V_0/V$ is the volume change occurring during consolidation, then

$$\frac{\Delta V_0}{V} = \frac{1-2\mu}{M} \times 3\sigma_3 \,, \tag{17}$$

or the unit volume change during the shear phase is

$$\frac{\Delta V}{V} = \frac{1-2\mu}{M} \times D. \tag{18}$$

By plotting ΔV/ versus D, as in Fig. 2b, and denoting the slope as

$$B_v = \frac{\Delta V/V}{D}, \tag{19}$$

then

$$B_v = \frac{1-2\mu}{M}, \tag{20}$$

and it is possible to determine Poisson's ratio by rearranging Eq. (16) into the form

$$\mu = \frac{1}{2} - \frac{B_v M}{2}. \tag{21}$$

Evaluation of Deformation Moduli

Observation of ESR versus ϵ_1 curves revealed a relationship consistent with that illustrated in Fig. 7. The ESR curves were nearly linear until minimum volume was reached, after which curvature began to increase rapidly. It is this behavior which led to the hypothesis that minimum volume represents a point similar to proportional limit.

Since that portion of each ESR curve prior to minimum volume was nearly linear, it was hypothesized that deformation moduli could be evaluated by using linear regression analyses rather than determining tangent or secant moduli. A linear regression program was thus developed for the IBM 360/65 computer.

Regression lines resulting from these anlyses indicated correlation coefficients ranging from 0.95 to 0.99, the majority of which were actually greater than 0.98. The assumption that intercepts of the regression lines were equal to zero was tested, using the "Student's t" distribution. At the significance level of $\alpha = 0.01$, this assumption was found valid in 84% of the tests. In the other 16%, the regression lines fit the data equally as well and deviation of the intercept from zero was thought to be at least partially attributed to initial seating conditions of the test specimens, porous stones, and loading piston. In every case, slopes of the regression lines appeared to be reasonable representations of the ESR versus ϵ_1 relationship existing prior to minimum volume.

The modulus for each material was evaluated by normalizing the regression values of B_m, Eq. (7). Observation of pore pressure data revealed that "u" was negligible with respect to the magnitude of the major principal stresses. Therefore, values of σ_3 were substituted into Eq. (7) rather than values of $\overline{\sigma}_3$ for this portion of the analyses. Deformation moduli for the materials studied are shown in Table 2 and graphically illustrated for material 1822 in Fig. 8.

Table 2. Regression analyses of ESR versus ϵ_1 curves.

Designation material	Mixing conditions	Asphalt cement content, %	σ_3	$B_m/100^a$	M, psi[b]	r[c]
1743	Field	4.27	10	22.21	22,200	0.999
			20	10.59	21,200	0.999
			30	6.70	20,100	0.999
			40	5.91	23,600	0.995
	Lab	4.06	10	13.10	13,100	0.999
			20	8.24	16,500	0.999
			30	5.72	17,200	0.997
			40	5.07	20,300	0.988
	Lab	4.97	10	10.47	10,500	0.999
			20	8.17	16,300	0.998
			30	4.85	14,500	0.994
1846	Field	3.80	10	12.58	12,600	0.989
			20	8.56	17,100	0.991
			30	5.34	16,000	0.997
			40	4.33	17,300	0.990
	Lab	4.05	10	15.16	15,200	0.990
			20	7.46	14,900	0.994
			30	5.52	16,600	0.989
			40	5.85	23,400	0.996
	Lab	4.98	10	15.22	15,200	0.991
			20	7.28	14,600	0.999
			30	5.94	17,800	0.997
			40	4.88	19,500	0.997
1822	Field	4.07	10	15.37	15,400	0.999
			20	8.71	17,400	0.999
			30	8.68	26,000	0.997
			40	5.79	23,200	0.997
	Lab	3.93	10	12.87	12,900	0.995
			20	3.78	7,600	0.996
			30	6.77	20,300	0.998
			40	6.22	24,900	0.991
	Lab	4.87	20	12.82	25,600	0.999
			30	7.22	21,600	0.999
			40	6.57	26,300	0.999
1855	Field	4.13	20	15.42	30,800	0.997
			30	10.49	30,300	0.996
			40	7.36	29,400	0.990
	Lab	4.30	10	18.77	18,800	0.992
			20	13.24	26,500	0.999
			30	11.12	33,300	0.999
			40	6.88	27,500	0.991
728	Field	4.47	10	14.38	14,400	0.997
			20	11.15	22,300	0.999
			30	6.62	19,800	0.985
			40	17.91	17,900	0.999
	Lab	4.49	10	9.12	18,200	0.945
			20	8.43	25,200	0.998
			30	7.74	31,000	0.993
1903	Field	5.33	10	11.20	11,200	0.999
			20	7.42	14,800	0.999
			30	7.10	21,300	0.998
			40	4.04	16,000	0.998

Table 2. Continued.

Designation material	Mixing conditions	Asphalt cement content, %	σ_3	$B_m/100$[a]	M, psi[b]	r[c]
	Lab	5.06	10	14.37	14,400	0.999
			20	8.80	17,600	0.996
			30	6.10	18,300	0.992
			40	4.58	18,300	0.986
1904	Field	3.80	10	16.73	16,700	0.996
			20	11.21	22,400	0.995
			30	6.86	20,600	0.999
			40	6.46	25,800	0.967
	Lab	4.09	10	12.39	12,400	0.997
			20	8.05	16,100	0.994
			30	5.69	17,100	0.985
			40	5.85	23,500	0.993
1485	Field	4.13	10	6.01	6,000	0.999
			20	6.00	12,000	0.997
			30	5.01	15,000	0.994
			40	3.04	12,000	0.992
	Lab	3.93	10	10.33	10,300	0.990
			20	5.66	11,300	0.994
			30	3.73	11,200	0.995
			40	3.26	13,000	0.991

[a] Slope of ESR curve divided by 100.

[b] Modulus of deformation.

[c] Correlation coefficient.

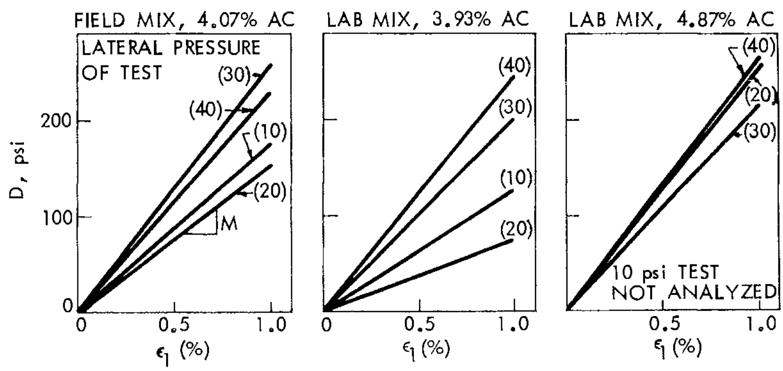

Fig. 8. Deformation moduli for material designation 1822.

Evaluation of M Versus σ_3

Stress-strain moduli for most soils will tend to increase with increasing confining pressure [4,5,13,14]. Taylor[5] stated that only a material with high intrinsic pressure (cohesion) could be expected to have a modulus that is essentially constant.

Table 3. Regression analyses of M versus σ_3.

Material designation	Mixing conditions	Asphalt cement content, %	Intercept[a]	Slope[b]	r[c]	S[d]	Sb[e]	$\overline{M}$[f]
1743	Field	4.27	210.0	0.31	0.269	17.560	0.785	21,800
	Lab	4.06	112.0	2.23	0.974	8.133	0.364	16,800
	Lab	4.97	97.7	2.00	0.674	31.027	2.194	13,800
1846	Field	3.80	125.0	1.30	0.771	16.971	0.759	15,800
	Lab	4.05	109.5	2.63	0.852	25.576	1.144	17,500
	Lab	4.98	127.5	1.61	0.909	11.677	0.522	16,800
1822	Field	4.07	125.0	3.20	0.837	33.136	1.482	20,500
	Lab	3.93	42.5	4.87	0.818	54.112	2.420	16,400
	Lab	4.87	234.5	0.35	0.138	35.518	2.511	24,500
1855	Field	4.13	322.7	–0.70	–0.987	1.633	0.115	30,200
	Lab	4.30	183.0	3.29	0.713	51.189	2.289	26,500
728	Field	4.47	134.3	2.70	0.669	42.458	3.002	18,800
	Lab	4.49	115.0	4.63	0.954	23.112	1.034	23,100
1903	Field	5.33	106 0	2.09	0.645	39.120	1.749	15 800
	Lab	5.06	140.5	1.24	0.859	11.666	0.522	17,200
1904	Field	3.80	150.0	2.55	0.869	22.995	1.028	21,400
	Lab	4.09	87.0	3.43	0.959	15.973	0.714	17,300
1485	Field	4.13	60.0	2.10	0.718	32.171	1.439	11,200
	Lab	3.93	94.5	0.80	0.916	5.523	0.247	11,400

[a] Intercept of regression analyses.
[b] Slope of regression analyses.
[c] Correlation coefficient.
[d] Standard deviation.
[e] Standard deviation of slope.
[f] Average modulus of deformation for each series of triaxial tests.

Since asphalt cement-treated granular materials are highly cohesive, this portion of the investigation analyzed the relationship between M and σ_3 using linear regression; testing the hypothesis that slopes of M versus σ_3, for any one material, were different from zero. Observation of moduli revealed a general tendency for M to increase slightly with increasing σ_3 for the 10 – 40 psi range of lateral pressures considered in this study (Fig. 8). It was also observed, that the M versus σ_3 relationship was not well defined. Slopes resulting from linear regression analyses verified the general trend of slight increases in M with increasing σ_3. But of the 46 cases analyzed, only 12 yielded correlation coefficients greater than 0.90 and 17 yielded correlation coefficients less than 0.70. A "t" test of the hypothesis that slopes of the plots were equal to zero revealed that at significance levels of both 0.01 and 0.05, the slopes could not be distinguished from zero. It therefore appears that for lateral pressures in the 10 – 40 psi range, the most significant values of the moduli are the mean values, $\overline{M}$, determined from each series of triaxial tests. Results of these analyses are shown in Table 3.

Evaluation of Poisson's Ratio

The values of Poisson's ratio in Table 4 were determined from Eq. (13), using the values of $\overline{\sigma_1}$, $\overline{\sigma_3}$, and $\Delta V/V$ at minimum volume, and the modulus, M, for each test. It can be observed that, although there may be variation in moduli with varying lateral pressures, the values of μ at minimum volume are fairly consistent. Poisson's ratio appears to be nearly the same for all materials, regardless of the type of aggregate, asphalt content, or gradation; a further substantiation of the hypothesized deformation model proposed in this investigation.

Plots of $\Delta V/V$ versus D to the point of minimum volume yielded the slope B_v of each material for use in Eq. (21). It was found that data from all tests in a series appeared to lie approximately along the same line, illustrated in Fig. 9 for material 1846. Linear regression analyses of the plots for all materials yielded correlation coefficients greater than 0.86, as shown in Table 5. Values of B_v determined in the linear regression analyses, and the mean values of deformation moduli, $\overline{M}$, previously determined, were used for calculation of μ shown in the right-hand column of Table 5.

Comparison of Poisson's ratios determined from Eqs. (13) and (21) (Tables 4 and 5 respectively) show surprisingly close results if considered in light of the various qualifying assumptions related to elastic theory and the many possibilities for actual test procedure error. In addition, the values of Poisson's ratio are indicative of

(1) the validity of the deformational model evaluated in this investigation, and (2) that volume change is primarily a function of deviator stress.

Table 4. Values of Poisson's ratio from Eq. (13).

Material designation	Mixing conditions	Asphalt cement content, %	μ at lateral pressure 10 psi	20 psi	30 psi	40 psi	$\bar{\mu}$[a]
1743	Field	4.27	0.304	0.404	0.431	0.412	0.387
	Lab	4.06	0.406	0.435	0.431	0.427	0.424
	Lab	4.97	0.438	0.418	0.449	–[b]	0.424
1846	Field	3.80	0.436	0.392	0.437	0.414	0.419
	Lab	4.05	0.439	0.391	0.443	0.433	0.426
	Lab	4.98	0.437	0.421	0.427	0.423	0.427
1822	Field	4.07	0.417	0.431	0.406	0.422	0.419
	Lab	3.93	0.392	0.453	0.426	0.427	0.424
	Lab	4.87	–	0.427	0.419	0.432	0.426
1855	Field	4.13	–	0.424	0.402	0.413	0.413
	Lab	4.30	0.415	0.425	0.420	0.446	0.426
728	Field	4.47	0.422	0.454	0.438	–	0.438
	Lab	4.49	0.452	0.415	0.431	0.423	0.430
1903	Field	5.33	0.452	0.414	0.427	0.434	0.431
	Lab	5.06	0.429	0.431	0.431	0.430	0.430
1904	Field	3.80	0.390	0.436	0.429	0.403	0.414
	Lab	4.09	0.422	0.440	0.443	0.414	0.429
1485	Field	4.13	0.445	0.450	0.454	0.454	0.446
	Lab	3.93	0.427	0.451	0.448	0.488	0.443

[a] Average value of μ from each series of triaxial tests.

[b] Dash indicates Poisson's ratio not determined.

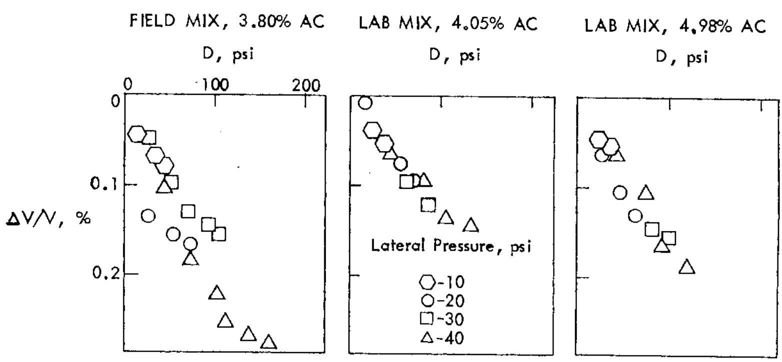

Fig. 9. Volume change versus deviator stress relationships for material 1846.

Table 5. Regression analyses of volume change versus deviator stress, plus Poisson's ratio, from Eq. (21).

Material designation	Mixing conditions	Asphalt cement content, %	B_v[a]	S_{Bv}[b]	r[c]	M[-d]	μ[e]
1743	Field	4.27	0.00101	0.00014	0.879	21,800	0.390
	Lab	4.06	0.00118	0.00009	0.965	16,800	0.401
	Lab	4.97	0.00125	0.00016	0.913	13,800	0.414
1846	Field	3.80	0.00163	0.00019	0.914	15,800	0.363
	Lab	4.05	0.00111	0.00009	0.967	17,500	0.403
	Lab	4.98	0.00143	0.00015	0.945	16,800	0.388
1822	Field	4.07	0.00104	0.00004	0.984	20,500	0.387
	Lab	3.93	0.00077	0.00012	0.863	16,400	0.373
	Lab	4.87	0.00057	0.00012	0.885	24,500	0.316
1855	Field	4.13	0.00094	0.00013	0.915	30,200	0.358
	Lab	4.30	0.00046	0.00005	0.937	26,500	0.439
728	Field	4.47	0.00078	0.00015	0.868	18,800	0.427
	Lab	4.49	0.00104	0.00021	0.881	23,100	0.380
1903	Field	5.33	0.00134	0.00015	0.909	15,800	0.394
	Lab	5.06	0.00134	0.00009	0.972	17,200	0.385
1904	Field	3.80	0.00103	0.00012	0.912	21,400	0.390
	Lab	4.09	0.00109	0.00016	0.905	17,300	0.406
1485	Field	4.13	0.00139	0.00022	0.872	11,200	0.422
	Lab	3.93	0.00159	0.00008	0.984	11,400	0.409

[a] Slope of $\Delta V/V$ versus deviator stress.

[b] Standard deviation of slope.

[c] Correlation coefficient.

[d] Average modulus of deformation for each series of triaxial tests.

[e] Poisson's ratio from Eq. (21).

Conclusions

By hypothesizing that a linear stress-strain relationship exists prior to minimum volume and that equations of elasticity are therefore applicable, this investigation led to the following conclusions regarding the shear strength and deformational characteristics of asphalt cement-treated granular materials.

1. Deformation during shear is characterized by an initial period of decreasing volume during which the stress-strain relationship is nearly linear.
2. Linear regression analysis appeared an appropriate tool for evaluating deformation moduli of asphalt cement-treated granular materials.
3. For the 10 – 40 psi range of lateral pressures studied, there was no well-defined relationship between deformation mod-

uli and lateral pressure. Although the data indicated a general tendency for slight increases in moduli with increasing lateral pressures, statistical analyses revealed that average moduli determined from a series of triaxial tests yielded the most significant values of moduli. The Poisson's ratio study also confirmed this conclusion.

4. Deformation moduli ranged from less than 10,000 to above 30,000 psi. Aggregates classified as limestone or dolomitic generally yielded the highest moduli, while gravels showed a trend towards the lower half of the range noted.
5. Poisson's ratio ranged from about 0.36 to 0.44.
6. Decreases in volume during the initial phase of shear deformation are approximately proportional to the deviator stresses acting upon an asphalt-treated granular material.

The practical application of deformation moduli derived from triaxial tests, to the design of flexible pavement base courses will depend upon the results of further investigations presently being conducted. However, the study reported herein indicates a possible use of deformation moduli as follows:

1. Evaluation of a large number of granular materials for use in development of coefficients of relative strength, utilized in the thickness design of flexible pavements.
2. Evaluation of the same materials as regards their potential rutting characteristics under traffic since volume change was shown to be primarily a function of deviator stress.

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References

McLeod, Norman W., "Application of Triaxial Testing to the Design of Bituminous Pavements," ASTM STP No. 106, *Triaxial Testing of Soils and Bituminous Mixtures,* 1951 (pp. 79-111).

The Asphalt Institute, *Mix Design Methods for Asphalt Concrete,* College Park Maryland, May 1963 (pp. 19-85).

Timoshenko, Stephen P., *History of Strength of Materials,* McGraw-Hill Book Company, Inc., 1953.

Terzaghi, Karl, *Theoretical Soil Mechanics,* John Wiley and Sons, Inc., New York, 1943.

Taylor, Donald W., *Fundamentals of Soil Mechanics,* John Wiley and Sons, Inc., New York, 1948.

Moore, William M., "Effect of Variations in Poisson's Ratio on Soil Triaxial Testing," Highway Research Record No. 108, HRB of National Research Council, Washington, D.C., 1966 (pp. 19-25).

Polakwski, N. H. & E. J. Ripling, *Strength and Structure of Engineering Materials,* Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1966 (pp. 203-212).

Richards, Cederic W., *Engineering Materials Science,* Wadsworth Publishing Company, Inc., Belmont, California (pp. 246-251).

Huang, Y. H., "Deformation and Volume Change Characteristics of a Sand-Asphalt Mixture under Constant Direct and Triaxial Compressive Stresses," April 1968.

Busching, H. W., W. H. Goetz, & M. E. Harr, "Stress Deformation Behavior of Anisotropic Bituminous Mixtures," Assoc. of Asphalt Paving Technology, Vol. 36, 1967.

Didukh, B. I. & V. A. Ioselevich, "Description of Deformation of a Soil Sample by Different Deformation Theories," Soil Mechanics and Foundations Engineering, No. 1, Jan.-Feb., 1966, Academy of Sciences of the USSR.

Hoover, J. M., *Factors Influencing Stability of Granular Base Course Mixes,* Final Report, Engineering Research Institute, Iowa State University, Ames, 1968.

Peck, Ralph B., Walter E. Hanson, & Thomas H. Thornburn, *Foundation Engineering,* John Wiley and Sons, Inc., New York, 1953.

Terzaghi, Karl & Ralph B. Peck, *Soil Mechanics in Engineering Practice,* John Wiley and Sons, Inc., New York, 1967.

Murphy, Glenn, *Properties of Engineering Materials,* International Textbook Company, Scranton, Pennsylvania, 1957.

Neville, Adam M. & John B. Kennedy, *Basic Statistical Methods for Engineers and Scientists,* International Textbook Company, Scranton, Pennsylvania, 1964 (257 pp.).

Timoshenko, S. & J. N. Goodier, *Theory of Elasticity,* McGraw-Hill Book Company, Inc., New York, 1951, 2*nd* edition.

Timoshenko, S. & D. H. Young, *Elements of Strength of Materials,* D. Van Nostrand Company, Inc., New York, 1962, 4*th* edition

Tschebotarioff, Gregory P., *Soil Mechanics, Foundations, and Earth Structures,* McGraw-Hill Book Co., Inc., 1951.