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EVALUATION OF MICROAGGREGATES BY SMITH TRIAXIAL TEST

by

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1.0 INTRODUCTION

Mineral aggregates used in bituminous concrete mixes are often subdivided into three groups: coarse aggregate, fine aggregate, and mineral filler. In this paper the term mineral filler has been changed to microaggregate which the authors feel is more descriptive for the following reasons:

1.1 All mineral particles used in bituminous concrete are usually pieces of rock differing in size.

1.2 They all have a positive and active role in regulating the engineering properties of bituminous concrete. The term "filler" suggests passivity.

1.3 The maximum diameter of mineral filler particles is generally set at 74 microns (passing 200 sieve). Because particles smaller than 100 microns cannot be seen by the average eye without magnification (microscope), the term "micro" is suitable.

Interest and research in expansion of knowledge in bituminous concrete materials and design has been growing rapidly during the last five years. This is necessary not only because of present traffic densities and loads but also to gather data for unavoidable future developments. Because bituminous concrete is a composite material, changes in various components produce different engineering properties. While research work has been done on effects of coarse aggregate, fine aggregate and microaggregate, the function and potentialities of the micro sizes are not well defined. Therefore, the work described in this paper was primarily concentrated in the area of microaggregates.

2.0 PURPOSE AND SCOPE OF THIS PAPER

The purpose of this study which is still continuing, is to determine the differences in rheological behavior of a given compacted mix in which the primary variable is the microaggregate.

The first research phase described in this paper involves seven microaggregates in a standard Massachusetts Class I mix. Kneading compaction simulating long term traffic densification on the road was employed to prepare triaxial specimens at differing asphalt contents. Test results include physical data on various components and mixes, and ultimate strength calculations using the logarithmic spiral method.

3.0 PROPERTIES OF MICROAGGREGATES EMPHASIZED IN TECHNICAL LITERATURE

Considerable effort has been expended in the past to determine the influential properties of microaggregates in a bituminous concrete mix. The overwhelming majority of the investigations have been concerned with physical characteristics of the particles. The following factors have drawn attention:

a) Particle size distribution
b) Particle shape
c) Particle surface texture
d) Unit surface area
e) Particle surface chemistry

Other factors, such as bulk density, fractional voids, pore diameter, permeability of packed fillers to fluids have also been mentioned.

3.1 Particle Size Distribution

Although gradation or particle size distribution is usually watched in selection of coarse and fine aggregate fractions, there are very few agencies which specify the required gradation of microaggregates. Research results indicate that one sized material, just passing the 74-micron sieve is inferior to finer and well graded particles, even close to the colloidal size range (4)* (12)*. The gradation might be especially important when the amount of fine and microaggregate exceeds a certain limit where the coarse aggregate particles become embedded in the finer aggregate and asphalt mass (10).

3.2 Particle Shape

The angularity of aggregates under certain conditions affects the resistance to deformation of a compacted mix (2) (9). The effect of particle shape of microaggregates upon the stability of mix appears to be greatest when the fine fraction (including microaggregates) is between 40 and 60 percent of the total weight of the mix (11).

3.3 Particle Surface Texture

Although the surface texture of larger particles (rock and sand) have some physical significance (31) it is not known what role the texture plays in particles below 74-micron size. Roughness of particles indicates larger surface area of the aggregate. Tensile strength might be superior with mixes containing aggregates with rough surfaces (42).

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*Numbers in parentheses refer to the number of reference in the selected Bibliography.

3.4 Surface Area

Surface area per given weight (or volume) of micro aggregate is often assumed as one of the most important physical properties influencing the mechanical behavior of a mix (9). Increased surface area can result in increased contact areas between particles thus influencing the strength characteristics of bituminous concrete (31).

3.5 Surface Chemistry

Mineral aggregates used in bituminous concrete are not inert particles but chemically active substances. Their composition varies widely presenting a problem of surface chemistry different for every rock used in a mix. Microaggregates have large surface area per unit mass and therefore their performance in a mix depends greatly upon their chemical composition. Adhesion of asphalt and selective adsorption of some components from the asphalt appear to be determined to a great degree by the surface chemistry of the microaggregate (50).

3.6 Miscellaneous Properties

Besides the considerations of basic properties of each microaggregate particle, attempts have been made to characterize their behavior in mass. For instance, the so-called "fractional void" content principle has been used to predict optimum binder content (6).

Several attempts have been made to study aggregates and their effect on mix properties without singling out any particular aspect (5) (13) (14) (15) (5). The most common conclusion is that various microaggregates can impart greatly different properties to the mix.

4.0 MICROAGGREGATES USED IN THIS STUDY

Altogether seven varieties of microaggregates were used in this initial study. Three of them were ground quartz, differing only in gradation. One was weathered quartz obtained from sand, similar in gradation to one of the freshly crushed quartz. In addition chrysotile asbestos, hydrated lime, and limestone microaggregates were tested. Brief descriptions of each microaggregate follows:

4.1 Freshly Ground Quartz

Ottawa sand was used to obtain coarse, medium and fine gradations of this microaggregate. Quartz is a hard mineral composed of silica and oxygen. Its crystals are usually prismatic and the particles hexagonal.

All of the coarsely ground quartz microaggregate passed sieve 200 and was retained on sieve 325. The medium fine quartz had a gradation between 5 and 74 microns, while the finely ground quartz contained particles down to colloidal size. The gradation curves are shown in Figure 2 while specific gravity and surface area data appear in Table 1.

4.2 Quartz Obtained From Sand Dust

The sand microaggregate was similar in gradation to the medium gradation quartz. Under the microscope the particles appeared to have lost some of their sharp edges and looked rounded as compared to the crushed particles.

4.3 Limestone

The limestone microaggregate was a commercial grade, containing primarily $Ca\ CO_3$ (high calcium limestone).

4.4 Hydrated Lime

Commercially available high calcium lime was used in this experiment. Chemically it consisted primarily of $Ca\ (OH)_2$.

4.5 Asbestos

This was a fibrous type material consisting of hydrous magnesium silicate. Because of its fibrous structure it is also called "chrysotile". The diameter of each fibre varied between 7.06×10^{-8} and 11.8×10^{-8} inch.

In a strict sense asbestos is not a typical microaggregate and its particle size distribution cannot be obtained by conventional methods. It is usually classified according to the Quebec Standard Test (Canadian Chrysotile Asbestos Classification).

Pertinent data on asbestos are given in Table 1.

5.0 TESTS AND EQUIPMENT USED TO CHARACTERIZE MICROAGGREGATES

Specific gravities of the various microaggregates (except asbestos) were obtained by the hydrometer test* after each material was passed through a 200 sieve.

The surface area was determined by the Air Permeability method using Bowen Apparatus.

*ASTM Designation D422-54T

6.0 OTHER BITUMINOUS CONCRETE COMPONENTS

Besides five percent of various microaggregates the Massachusetts Class I Top Mix had the following components (for aggregate gradation see Figure 1):

Coarse Aggregate: Massachusetts rhyolite, a fine-grained hard crushed rock with low absorption characteristics and apparent specific gravity of 2.66.

Fine Aggregate: Clean quartz sand with a small amount of mica, apparent specific gravity of 2.63.

Asphalt: 85-100 penetration, 5.5 to 7.5 percent by weight of the total mix. Massachusetts specification calls for asphalt range between 6.0 and 7.0 percent.

7.0 TRIAXIAL EVALUATION TEST CHOSEN

The Smith triaxial test was selected for the comparative strength tests described in this paper. This is the first step which will be followed by tension studies using various temperatures and times of loading. The primary reasons for selecting this test were:

a) the test gives comparative angle of internal friction and cohesion values which can be used for shear strength calculations.

b) the test is relatively simple and the apparatus was readily available.

The triaxial cell is described in detail in references 34 and 40.

8.0 TRIAXIAL SPECIMEN PREPARATION AND TESTING

The specimens for triaxial testing were prepared one at a time. Following is a brief outline of the procedures and the reasons for them:

8.1 Heating and Mixing

Aggregates were first weighed out combining various sieved particle sizes. The total weight of aggregates for any specimen was kept constant while the asphalt content was varied.

The aggregates and the 85-100 penetration asphalt were heated to 300F and then combined by a mechanical mixer. The mixing time was kept at a constant two minutes.

8.2 Compaction

The Smith triaxial method calls for static compaction of the eight-inch specimens. In order to measure what happens to a Massachusetts Class I Top Mix under heavy compaction effort, as it might be experienced during the life of the pavement which is rolled over by frequent and heavy traffic loads, the kneading compaction was chosen (1) (31) (32).

The method of compaction for Hveem stability specimens was taken as a guide. Each eight-inch specimen was made in six layers of about equal thickness. The unit compaction foot pressure was taken as 500 psi. The energy of densification used for Hveem's specimens amount to 150 applications of 500 psi. pressure while for the triaxial specimens a total of 450 tampings at 500 psi. were used (because the triaxial specimens have about three times as large a mass).

In order to keep the temperature of the specimen above 240F during compaction, the molds and the tamping foot were heated (see Figure 3). Thus by controlling the temperature, using 500 psi. unit pressure, and layers of about 1 1/2 inches in thickness, homogeneous, well compacted specimens were obtained.

After molding, all specimens were cooled, taken out of the mold, and placed in a special holder (too keep their shape) for forty-eight hours of curing.

8.3 Testing

After curing at 77F ($\mp$ 2), the specimens were placed in the triaxial cell and subjected to a load. The equipment and procedures for the Smith triaxial cell are described in references 34 and 40. Briefly, this test is a closed system triaxial test where the vertical loads are applied by increments. The vertical pressure is plotted against the corresponding horizontal pressure and a tangent is drawn to the curve. From these data the angle of internal friction and cohesion is obtained (see Example in Figure 5).

9.0 STRENGTH ANALYSIS OF MIX USING LOGARITHMIC SPIRAL

From the triaxial test values of friction and cohesion are obtained, both of which contribute to the strength of bituminous concrete under load. In order to obtain comparative values of shear strength for the various mixes, the logarithmic spiral method and ultimate strength concept were used (50). The lengthy calculations were done with an electronic computer.

10.0 RESULTS

The results of this investigation are summarized in various tables and figures. These include mix data, triaxial test measurements and ultimate strength calculations.

11.0 DISCUSSION OF TEST RESULTS

Following is a discussion of various observations and measurements on the microaggregates and mixes in which they were incorporated:

11.1 Observations During Mixing and Compaction

Although the visual appearance of the various compositions during and after mixing was not the same, the differences between the mixes containing granular microaggregates (at a given asphalt content) were generally minor. The mixes containing lime seemed to have a lower mass viscosity after mixing. Asbestos mixes looked noticeably dryer than the rest, especially at 5.5 percent binder content, when it became difficult to coat the coarse aggregate particles. This was one of the reasons why asphalt contents below 5.5 percent were not used in the test series.

At asphalt contents approaching 7.5 percent all mixes with granular microaggregates showed flushing of the asphalt on the top of the surface. The 50-50 mixture of asbestos and limestone (ALS)* showed only traces of flushing while the five-percent-asbestos mixes had none.

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*ALS - abbreviation for asbestos-limestone mixes as indicated in Table 1.

11.2 Void Contents of the Mixes

Amount of voids at various asphalt contents is shown in Figure 9. The asbestos (A) has the highest void content with other curves being concentrated in a relatively narrow band. Because the basic mix proportions and compaction efforts are similar in all cases, the indications are that mixes containing just asbestos (five percent) as microaggregate would require more compactive effort to obtain a certain density at a given asphalt content. As soon as 50 percent of the asbestos was replaced by limestone, the void content was lowered close to other granular microaggregates. The straight limestone mixes had very low voids.

The void volume in most mixes at asphalt contents of six percent or higher were near the one percent mark or below.

11.3 Angle of Internal Friction

The curves for angle of internal friction are given in Figure 6. As expected, the frictional resistance of a mix decreases with increased asphalt content. Lime (HL), fine ground quartz (FQ), and limestone (LS) microaggregates in general show the lowest friction angles. Asbestos is characterized by a relatively gradual slope as compared to the others. From these data it appears that fine graded granular microaggregates with relatively large surface areas act more as viscosity boosters of the asphalt and do not help much to hold the angle of internal friction with increasing asphalt contents (compare HL, FQ and LS in Figures 2 and 6). Microaggregates containing more coarse particles (close to 74 microns) appear to make the mix less sensitive to asphalt changes. An exception here is asbestos which has a large surface area (and a fibre shape particle). All asbestos mixes looked drier during mixing and compaction as compared to the others. This may be due to both the larger surface area and to greater chemical affinity of asphalt for the asbestos surfaces.

The limestone microaggregate mix had a very high angle of internal friction at 5.5 percent asphalt content. When a combination of asbestos and limestone was used the friction angle still remained high and above that of pure asbestos at this low asphalt content. It decreased at a more rapid rate with higher asphalt percentages.

11.4 Cohesion

The cohesion of the mixes varied widely at the 5.5 percent asphalt (see Figure 7). At six percent and higher asphalt contents the cohesion was close to or below 10 psi. except for asbestos (A) and asbestos-limestone (ALS).

The high cohesive strength of the asbestos mixes cannot be fully explained with the facts available. The fibrous particles may act as a reinforcement in the asphalt. The affinity of asphalt for asbestos can result in excellent bonding, thus adding to the reinforcement.

11.5 Satisfactory Mixes

Although the ultimate strength analysis was used in this paper to combine the angle of internal friction and cohesion there are no agreed upon criteria available to define a satisfactory mix by this method. In order to get another indication of which mixes are satisfactory for highway use, a chart from the Smith triaxial design procedure in reference 34 was used. The results of this comparison are given in Table 2. According to these criteria the asbestos mix is satisfactory for all asphalt contents tested; the asbestos limestone mix for all but the 7.5 percent; while the rest of the mixes for 5.5 or 6.0 percent only.

11.5 Ultimate Strength of Various Mixes

The ultimate strength curves obtained from combining cohesion and frictional strength are given in Figure 8. In this comparison most microaggregates had high strength at 5.5 percent asphalt. This strength, decreased as the asphalt content increased, but with different rapidity. With limestone (LS), lime (HL) and fine quartz (FQ) mixes the slopes of the curves are very steep dropping to 100 psi. strength at 6.5 percent asphalt content.

The limestone-asbestos (ALS) mixes still had relatively high ultimate strength as compared with the others. Only at 7.5 percent binder content the curve joins other microaggregates. This combination has reduced the slope of the curve and raised the strength of the mix considerably.

Much of what has been said about the factors affecting the angle of internal friction and cohesion values can be also applied to the ultimate strength curves. The mixes containing finely graded microaggregates lose their strength very rapidly with increases in asphalt contents from 5.5 to 7.5 percent. The mixes containing asbestos also lose their strength but at a slower rate. In addition, their initial strength at 5.5 percent binder content is relatively high.

Mixes containing just asbestos microaggregate had generally a higher void content (see Figure 9). Although low voids are often associated with low strength of a compacted mix, the results show that strength is not ruled by the volume of voids alone. For instance, asbestos (A) and limestone (LS) mixes at 5.5 and 7.5 percent asphalt content respectively show reasonably high strength at about 0.5 percent void content.

11.6 Possible Trends of Ultimate Strength at Asphalt Contents Below 5.5 Percent

The asphalt contents used in this investigation were taken from the specification, Massachusetts Class I Top mix, extending the range half a percent on each side of the 6 to 7 percent limits. The aggregate in asbestos mixes at 5.5 percent binder content were difficult to coat and the mix looked dry. Also, the ultimate void content was over five percent. Therefore, it is very doubtful that this mix could be compacted well on a road at 5.5 percent or lower.

The other mixes, especially the lime and limestone, probably could be mixed and compacted by a kneading compactor at lower asphalt contents than reported in this paper. It is not known whether on the road these mixes can be properly compacted to a durable pavement surface at asphalt contents below 5.5 percent.

If the testing would be continued, however, at lower asphalt contents, a point of maximum ultimate strength would be reached.

11.7 Practical Implications

If the compaction technique and strength measurements in this work are close to actuality, the ultimate strength curves reveal a significant finding as far as the basic mix used in this study is concerned. (Massachusetts Class I Top mix). All mixes containing various microaggregates, except those of asbestos show very low ultimate strength at seven percent asphalt content which is allowed according to the specifications. This explains some cases of unstable pavements found in Massachusetts. The present specified maximum asphalt content of seven percent is too high for the mix with granular microaggregates similar to those used in the tests. Under heavy traffic with the passing of time such mixes can lose their strength.

The desirable maximum asphalt content varies with different microaggregates. For coarse and medium gradation quartz this limit may be 6.5 percent or slightly below. For limestone, finely graded quartz, and hydrated lime the maximum asphalt content should not exceed six percent. If asbestos is added to the limestone microaggregate (50-50 mixture), the present seven percent limit seems reasonable. For straight asbestos microaggregate the maximum "allowable" asphalt content is beyond the 7.5 percent mark.

12.0 CONCLUSIONS

These conclusions pertain to the bituminous concrete mixes, test procedures and assumptions described in this paper:

12.1 Kneading compaction, simulating ultimate densification of the mix by traffic on the road, produced low void contents at the minimum specified (six percent) amount of asphalt with all mixes containing granular microaggregates. Mixes with fibrous (asbestos) microaggregate averaged a higher volume of voids at the asphalt content investigated.

12.2 The angle of internal friction of various mixes did not differ greatly at the lowest asphalt content (5.5 percent). The decline of this angle with increasing asphalt content was less rapid with the mix containing straight asbestos microaggregate.

12.3 The cohesion as measured by the Smith triaxial test differed widely at the low (5.5 percent) asphalt content. It decreased rapidly with increasing asphalt content in the case of granular microaggregates but remained relatively high with the fibrous (asbestos) microaggregate mix.

12.4 The ultimate shear strength values (using logarithmic spiral analysis) were high with all mixes at 5.5 asphalt content. With increasing asphalt contents, the mixes containing relatively fine graded microaggregates showed sharper decline in strength than mixes having coarse more uniform microaggregate particles. The five-percent-asbestos mix showed the highest strength leaving the asbestos-limestone mix in the second place.

12.5 It is suggested that Massachusetts Class I Top mix used for heavily travelled roads should be either re-designed or the maximum allowable asphalt content lowered to 6.5 or even 6.0 percent when granular microaggregates (such as tested in this series) are used. When asbestos was added replacing

2 1/2 percent of the limestone microaggregate, the mix had still fairly high strength at asphalt content of 7.0 percent.

Similarly, when the whole five percent of microaggregate was asbestos, the ultimate strength even at 7.5 percent of asphalt was relatively high.

13.0 FUTURE WORK

Research work is in progress to evaluate the various mixes at temperatures between-20 and 140F and loading times of 1/1000 second to 1000 hours.

ACKNOWLEDGEMENTS

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TABLE 1

BASIC DATA ON VARIOUS MICROAGGREGATES

Name	Code Letters	Specific Gravity	Surface Area sq.cm. per Gram
7M Asbestos (Canadian Chrysotile)	A	2.640	7250
Limestone	LS	2.730	2500
Coarse Quartz	CQ	2.649	1605
Medium Gradation Quartz	MQ	2.648	1670
Fine Gradation Quartz	FQ	2.647	4580
Sand Dust (Quartz)	SQ	2.651	1450
Hydrated Lime	HL	2.617	1810
7M Asbestos, 50% Limestone, 50%	ALS	-	4875

TABLE 2

SATISFACTORY MIXES USING ANGLE OF INTERNAL FRICTION AND COHESION AS OUTLINED IN PUBLICATION 34, PAGE 122

Microaggregate	Percent of Asphalt 5.5	6.0	6.5	7.0	7.5
Asbestos	X	X	X	X	X
Asbestos-Limestone	X	X	X	X	
Coarse Quartz	X	X			
Medium Fine Quartz	X	X			
Sand Dust (Quartz)	X	X			
Limestone	X				
Lime	X				
Fine Ground Quartz	X				

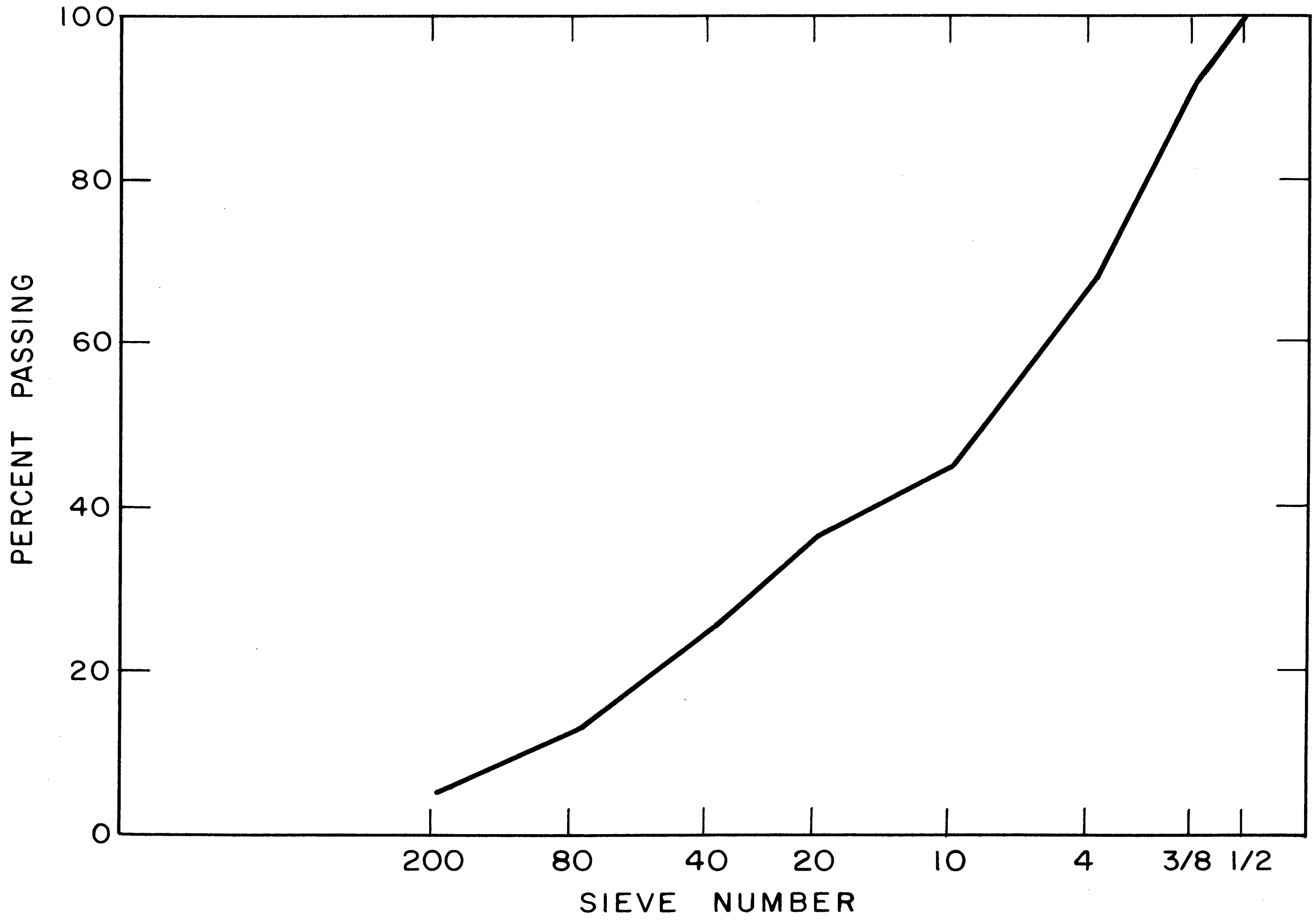

Figure I. Average gradation curve of Massachusetts Class I Top Mix, used as a basis in this investigation.

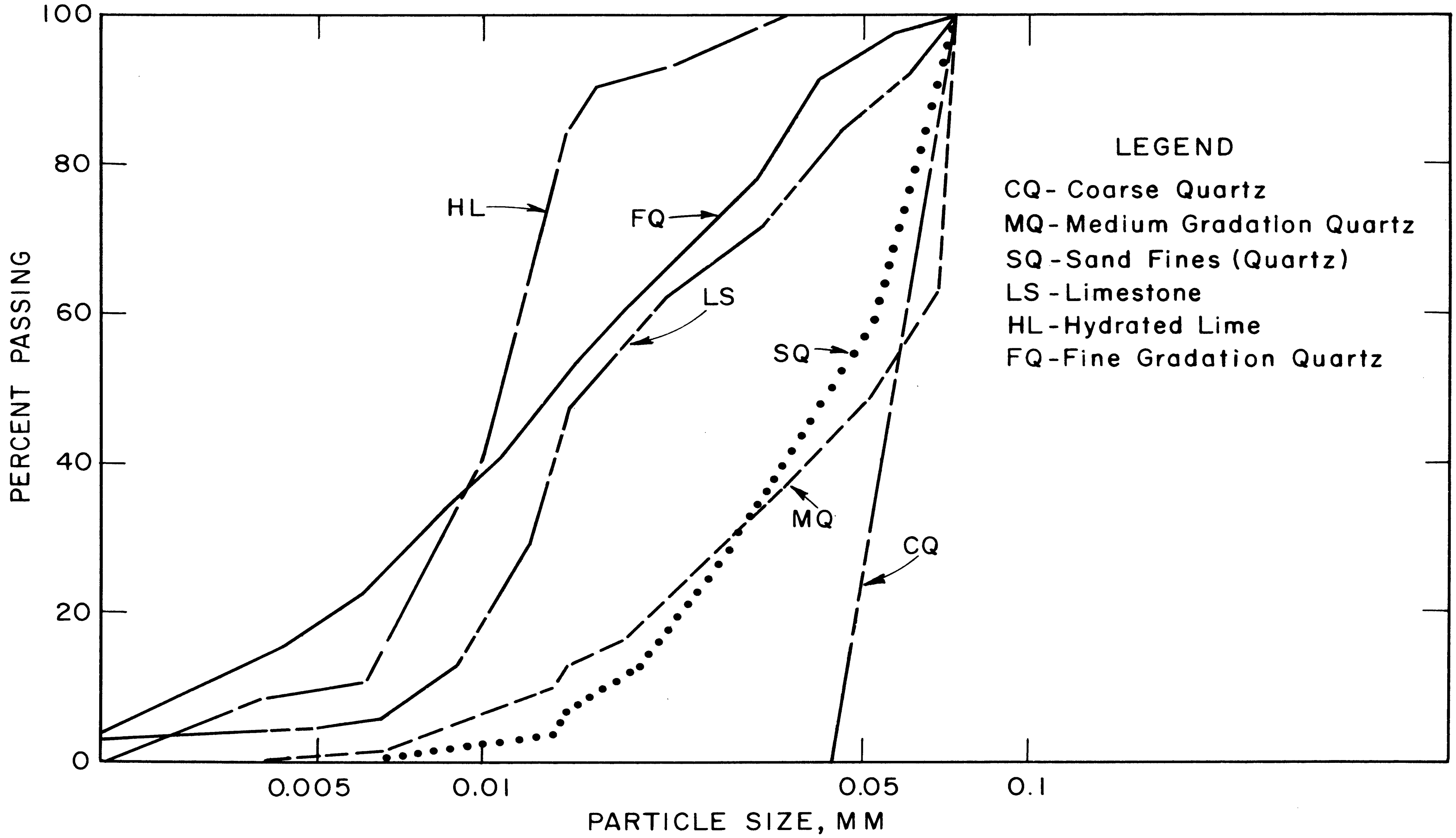

Figure 2. Particle size distribution of various microaggregates as indicated by hydrometer test. Asbestos not included.

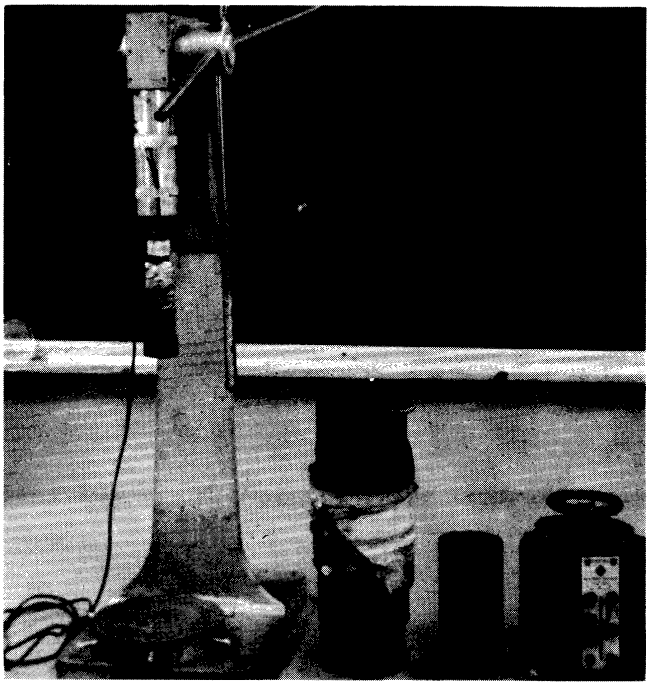

Figure 3. Manual kneading compactor with assembled triaxial test mold and accessories for temperature control.

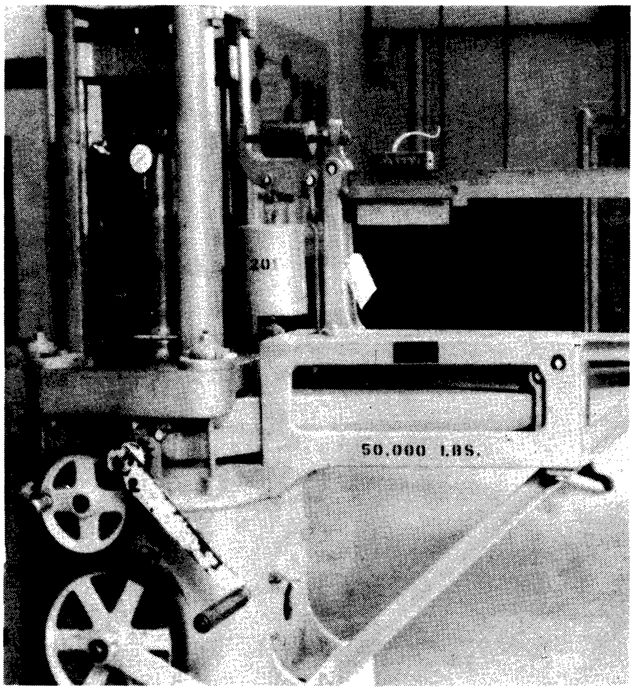

Figure 4. Loading machine with Smith triaxial test cell in place.

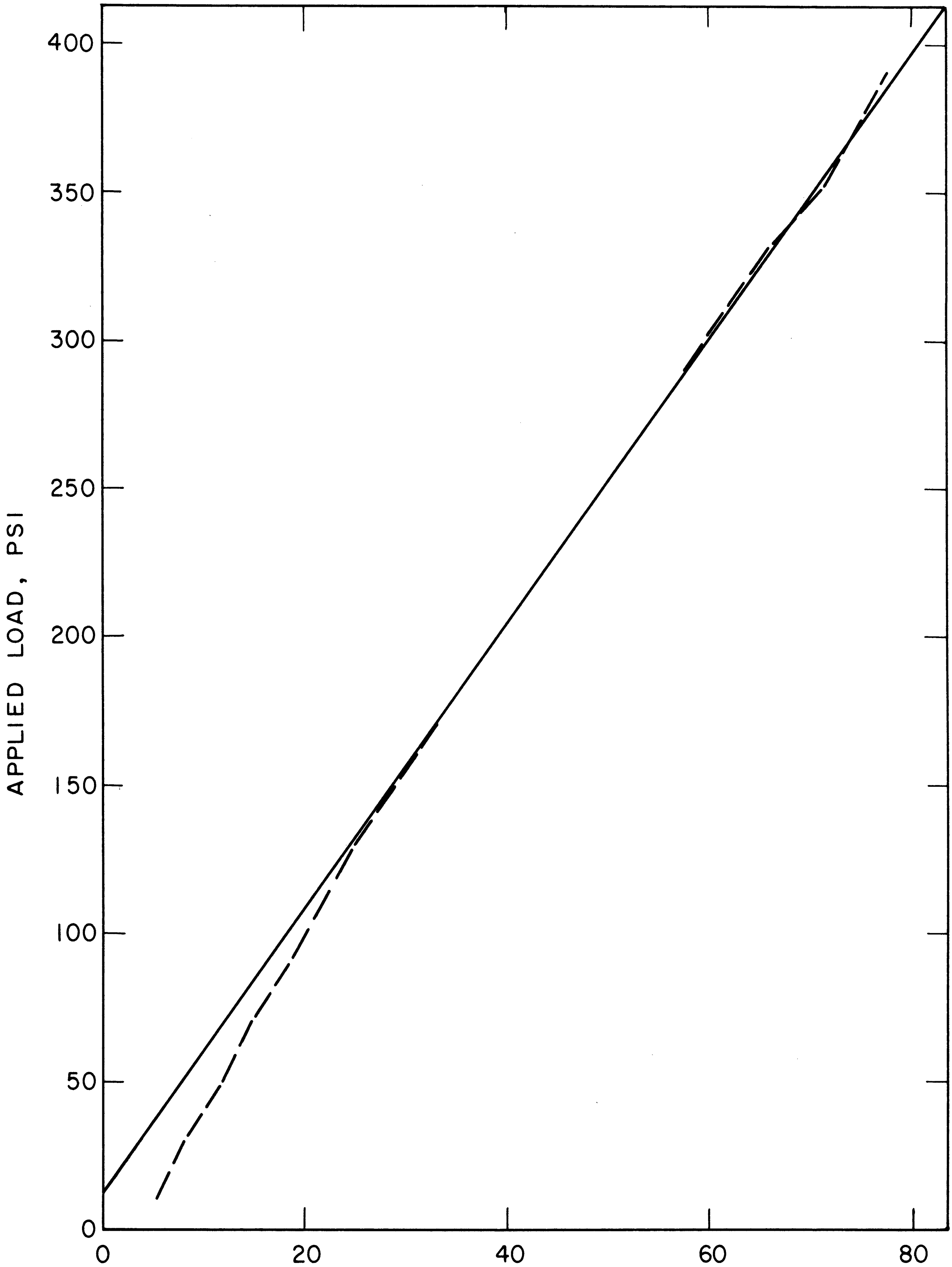

Figure 5. An example of triaxial test curve from which cohesion and friction values are obtained.

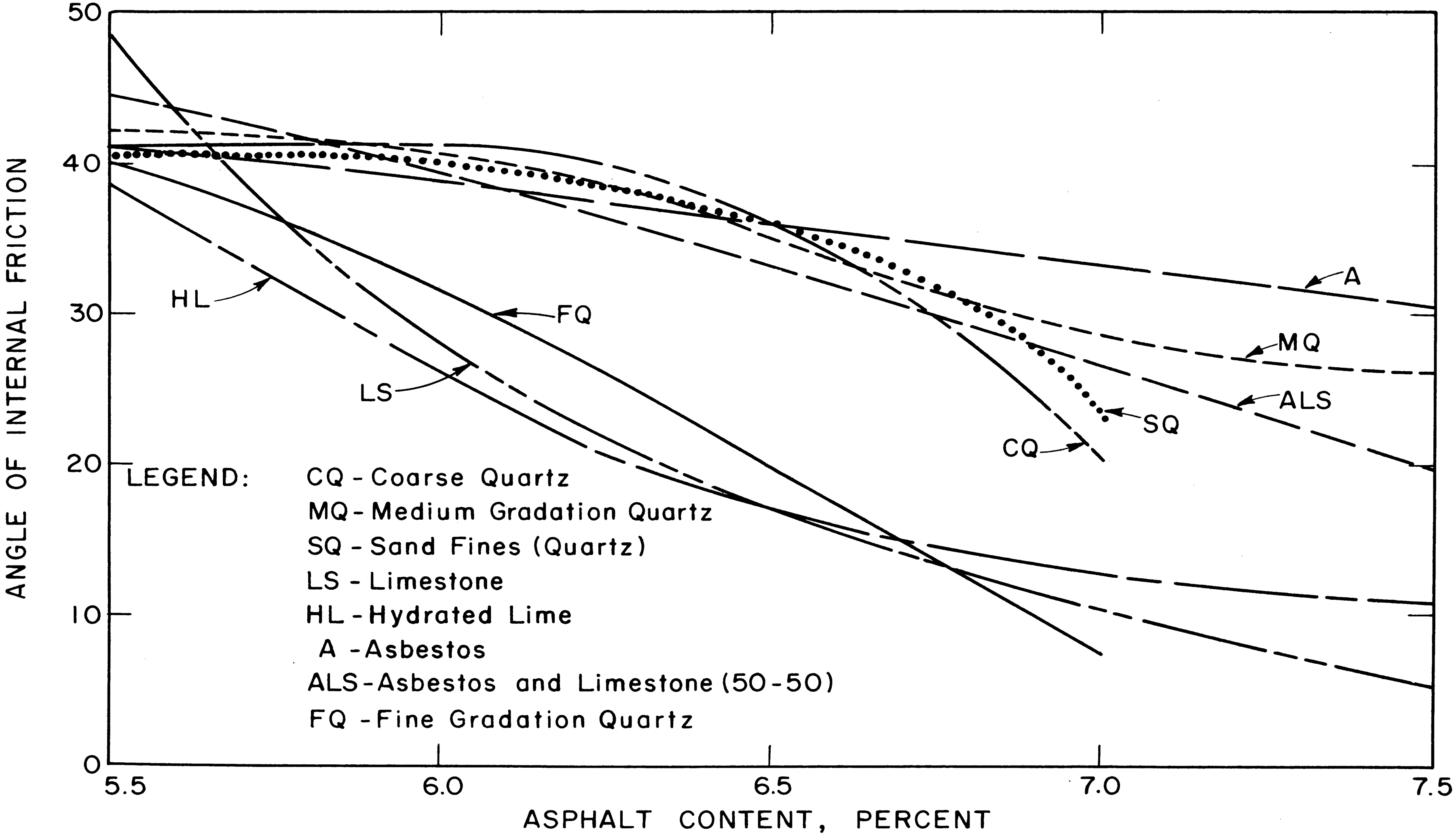

Figure 6. Asphalt content versus angle of internal friction for a given mix with five percent of various microaggregates.

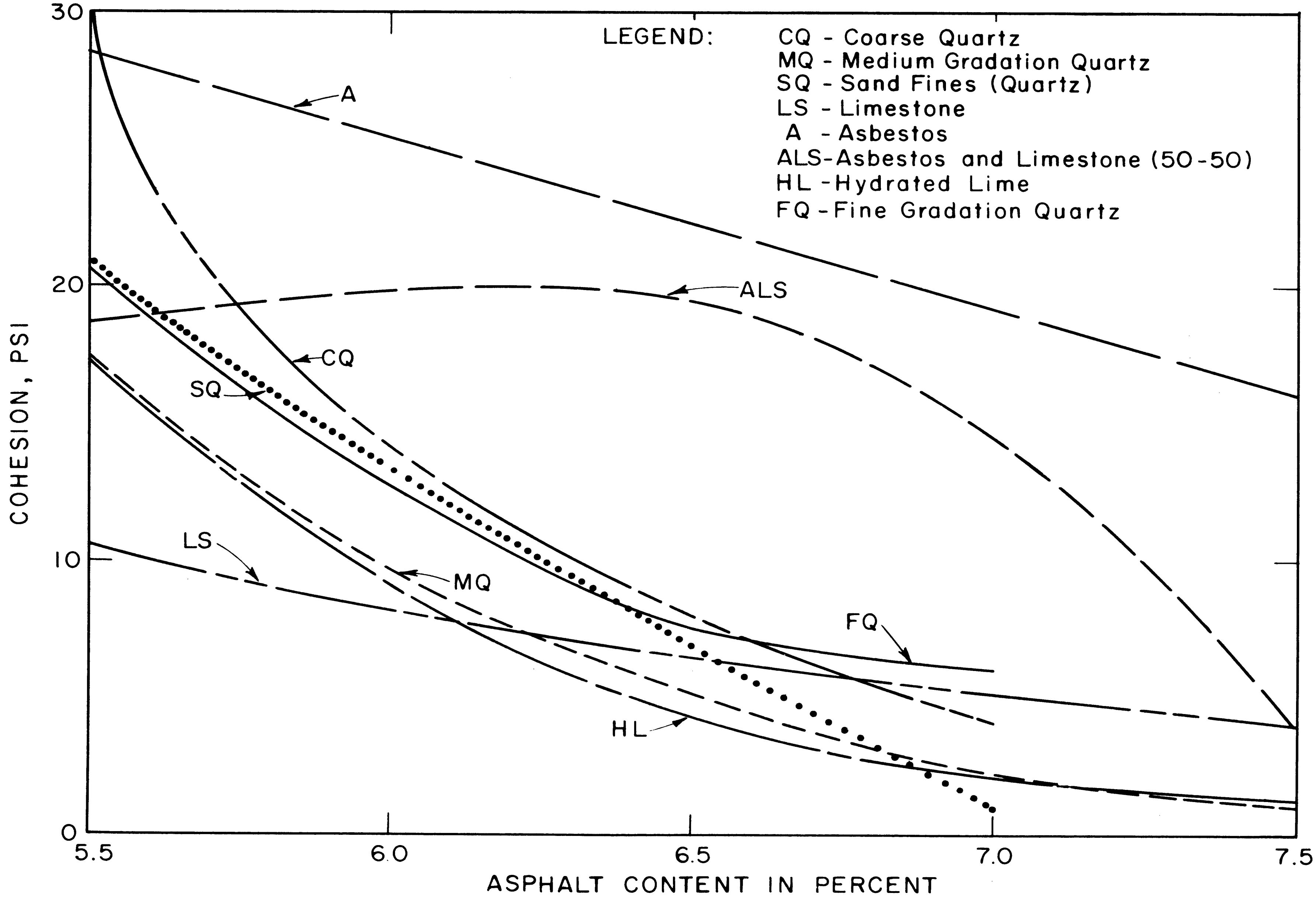

Figure 7. Asphalt content versus cohesion for a given mix with five percent of various microaggregates.

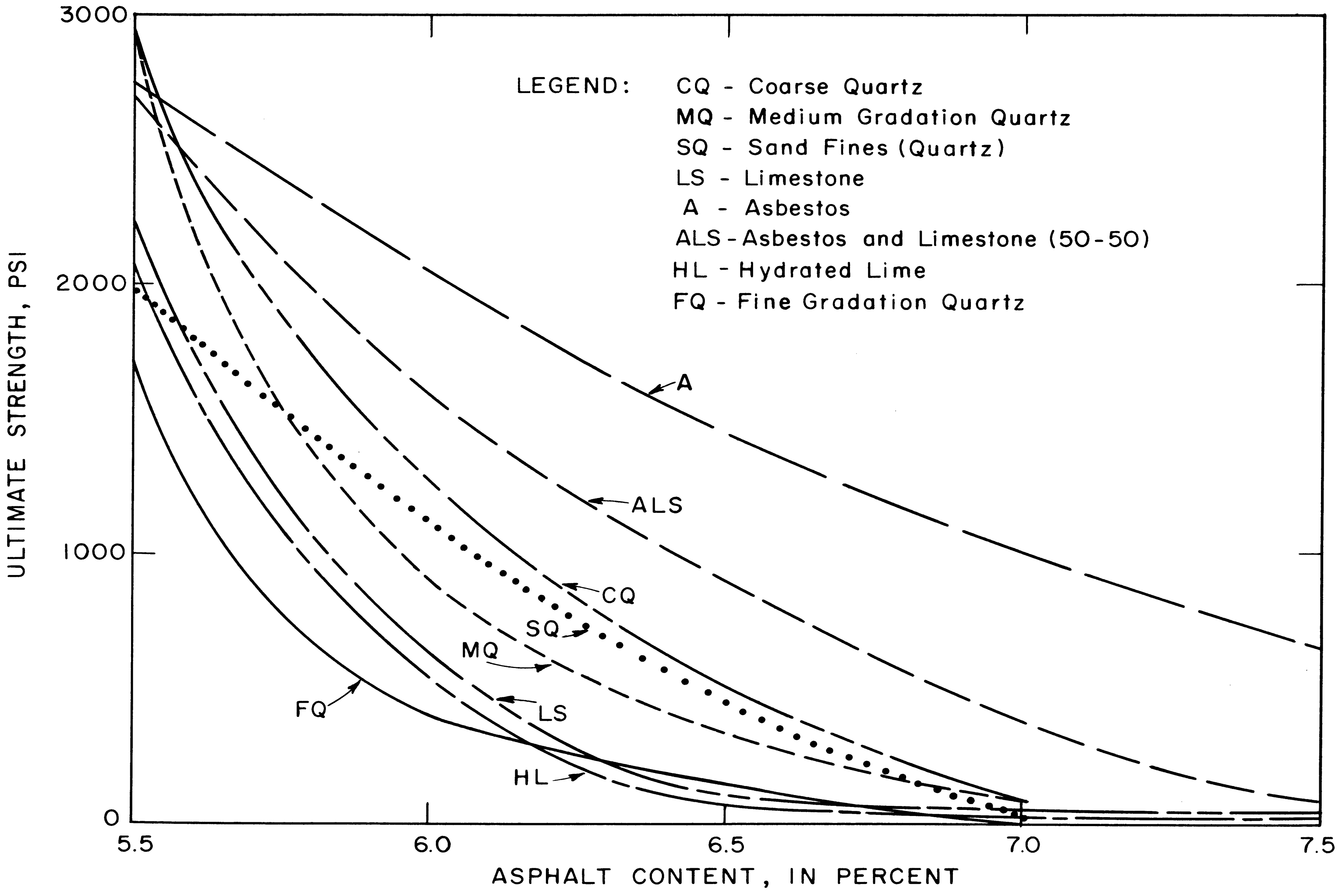

Figure 8. Ultimate strength of mixes containing various microaggregates and at various asphalt contents.

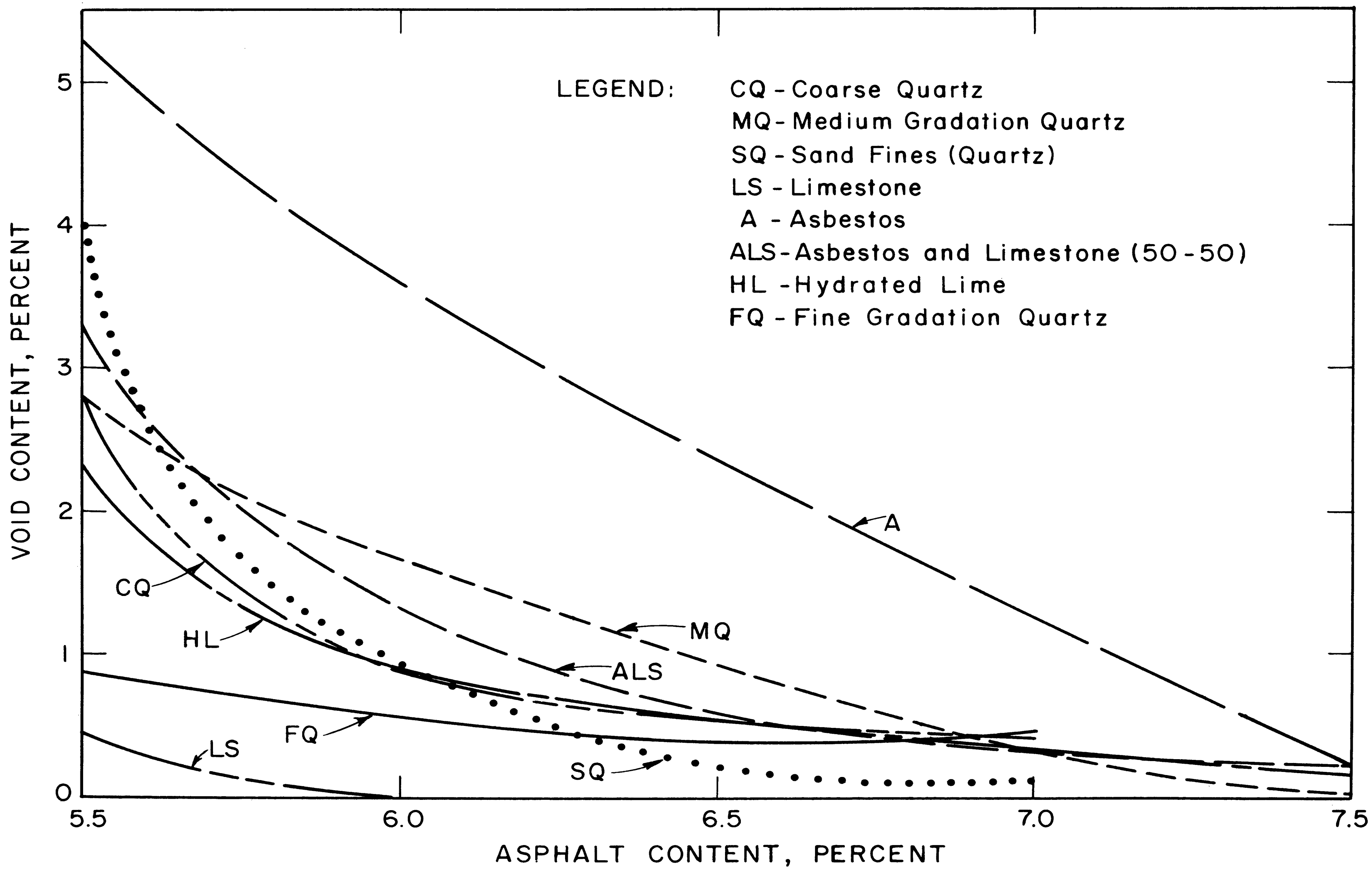

Figure 9. Voids versus asphalt content for various mixes.

APPENDIX

Basic Data Used to Obtain the Various Figures and Tables in this Paper.

Asphalt Content, Percent	Specific Gravity, Aggregate	Bulk Density, Specimen	Maximum Theoretical Density	Percent Voids	Friction Angle	Cohesion Psi
		Hydrated Lime Mixes (HL).				
5.5	2.617	2.351	2.410	2.49	38.8	24.9
5.5	2.617	2.360	2.410	2.08	37.8	30.4
6.0	2.617	2.368	2.389	0.88	22.4	9.9
6.5	2.617	2.355	2.373	0.72	18.2	4.7
6.5	2.617	2.360	2.374	0.35	16.6	4.3
7.0	2.617	2.342	2.355	0.65	12.0	3.5
7.5	2.617	2.352	2.351	0	5.2	1.4
		Limestone Mixes (LS).				
5.5	2.653	2.425	2.435	0.41	51.5	9.5
5.5	2.653	2.421	2.434	0.52	45.2	11.7
6.5	2.653	2.378	2.399	0.41	18.2	5.3
6.5	2.653	2.401	2.398	0	16.0	7.4
7.5	2.653	2.384	2.380	0	10.8	4.1
		Asbestos Mixes (A).				
5.5	2.641	2.298	2.427	5.29	40.7	28.4
6.0	2.641	2.307	2.407	4.15	36.2	25.1
6.5	2.641	2.313	2.390	3.22	37.2	22.3
7.0	2.641	2.337	2.372	1.48	31.2	17.7
7.0	2.641	2.348	2.373	1.05	36.4	21.5
7.5	2.641	2.351	2.356	0.21	30.2	16.0

APPENDIX (Continued -2)

Asphalt Content, Percent	Specific Gravity, Aggregate	Bulk Density, Specimen	Maximum Theoretical Density	Percent Voids	Friction Angle	Cohesion psi
			Asbestos-Limestone Mixes (ALS).			
5.5	2.646	2.365	2.430	2.65	48.6	18.0
5.5	2.646	2.335	2.431	3.96	42.0	20.7
6.5	2.646	2.381	2.395	0.56	28.4	16.8
6.5	2.646	2.380	2.395	0.58	37.0	22.0
7.5	2.646	2.354	2.360	0.25	22.0	3.2
7.5	2.646	2.357	2.360	0.10	17.0	4.4
			Medium Graded Quartz (MQ).			
5.5	2.648	2.365	2.433	2.79	41.8	17.9
6.0	2.648	2.373	2.413	1.66	40.4	9.3
6.5	2.648	2.375	2.396	0.88	35.0	5.3
7.0	2.648	2.371	2.378	0.30	28.6	2.1
7.5	2.648	2.363	2.363	0	30.0	1.4
			Coarse Graded Quartz (CQ).			
5.5	2.650	2.365	2.434	2.84	40.6	30.5
6.5	2.650	2.383	2.391	0.59	36.0	7.7
7.0	2.650	2.369	2.379	0.42	20.6	4.3
			Fine Graded Quartz (FQ)			
5.5	2.648	2.414	2.434	0.81	39.4	20.0
6.0	2.648	2.397	2.413	0.66	33.0	10.0
6.5	2.648	2.385	2.395	0.42	19.4	7.3
7.0	2.648	2.367	2.378	0.46	7.5	5.2