Stabilization of Tennessee Gravel and Chert Bases

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This paper reviews laboratory investigations of stabilization of base materials. Most of the tests have been performed on four materials, two cherts and two gravels, in combination with some or all of the following admixtures: portland cement, cutback asphalt (RC-2), emulsified asphalt, road tar (RT-5), hydrated lime, lime and flyash, calcium acrylate, and calcium chloride.

The report presents information in two categories: (1) comparison of the effectiveness of the several admixtures in stabilizing the soils included in the study and (2) comparison of the value of a number of possible methods of test in differentiating between the suitability of the materials, with or without admixtures, for use in base construction.

It is concluded that any of the materials tested may be adequately stabilized by the addition of small percentages of portland cement. Quantities in excess of 4 percent by weight are not recommended. It is also shown that some of the soils may be stabilized to advantage with certain types of asphalt.

The report also concludes that of the various tests employed throughout the study, the triaxial-compression test appears to be the most promising in evaluating the usefulness of coarse granular materials. Attention is directed to a procedure which may evolve into a satisfactory design method. It is pointed out, however, that the full effectiveness of the test and data collected cannot be utilized until these laboratory investigations are supplemented by equally extensive field investigations in the nature of test sections. The construction of such sections is contemplated in the near future.

● FOR nearly 3 years the Tennessee Highway Research Program, a cooperative organization of the Tennessee Department of Highways and Public Works and the University of Tennessee, has been actively engaged in a study of the stabilization of gravel and chert bases. This study was first suggested because there are large areas of Tennessee which do not have available sufficient quantities of good base material. It was hoped that some means might be developed for stabilizing such poorer materials as might be locally available.

As the study has progressed it has more or less divided itself into two phases. One is a study of the changes which occur in the quality of a base material when it is combined with one or more of a number or admixtures. The other is a search for a method of test which adequately defines those properties of the material which determine its usefulness in base construction.

After some preliminary tests on a number of small samples, four soils were selected for use in this study. Two were cherts obtained from Benton County and two were bank gravels obtained from western Tennessee. On the basis of the original samples tested it was expected that one chert and one gravel would be classified as good base materials and one of each classified as a poor base material. When the larger quantities of these materials were obtained, however, approximately 5 tons of each being brought into the laboratory, it was found that there was relatively little difference between the two cherts. These have been identified as Soils 10 and 11, with the poorer chert designated as Soil 10. The two gravels have been identified as Soils 12 and 13 with the poor gravel designated as Soil 12.

The principal properties of the four soils are indicated by the test values shown in Table 1. The grain-size distribution of the two cherts (Samples 10 and 11) and of the two gravels (Samples 12 and 13) are indicated in Figures 1 and 2.

In general, the attempts to stabilize these soils have been made by adding some more or less commercially available admixture to the soils tested. Admixtures included to date have been portland cement, cutback asphalt (RC-2), emulsified asphalt, road tar (RT-5), hydrated lime, lime and flyash, calcium acrylate, and calcium chloride.

All of the admixtures mentioned above, with the exceptions of the lime and flyash and the calcium acrylate, are well known and readily available materials which are widely used in road construction. The two exceptions were included in this study because it was felt that they held special promise. The former is a waste material of lime production. It contains about 47 percent $Ca(OH)_2$, 26 percent $Ca(CO)_3$, and something over 13 percent finely divided carbon. Since combinations of lime and flyash have been successfully used for stabilization of certain types of bases and subgrades in the past, it was felt that this material should be investigated. Its nature, that of a waste product, is such that it would be extremely cheap if it were satisfactory for the use desired.

The second exception, calcium acrylate, is an experimental material produced by the Rohm and Haas Company in Philadelphia, Pennsylvania. It has been and is being extensively investigated by other organizations, notably the Massachusetts Institute of Technology, on behalf of the armed forces. It has been reported to impart to fine-grained soils such properties as tensile strength, flexibility, and resilience. It is rather expensive at the present time, but should it prove satisfactory for base stabilization and its use increase, mass production would doubtless lower the unit cost considerably. It was felt that some of the unusual properties attributed to this material warranted its inclusion in this study.

Several different tests were employed

TABLE I

PHYSICAL PROPERTIES OF SOILS NOS. 10, 11, 12 and 13

	Soil No 10	Soil No 11	Soil No 12	Soil No 13
Liquid Limit (%)	24.6	23 3	32 0	19 8
Plastic Limit (%)	17 2	NP	19 5	NP
Plasticity Index (%)	7 4	NP	12 5	NP
Shrinkage Limit (%)	18 1	20 5	15 6	20 2
Specific Gravity, — No 4 Fraction	2 62	2 63	2 68	2 68
Optimum Moisture Content, — No.4 Fraction (%) (Standard Proctor)	13 4	11 0	11 0	15.0
Maximum Density, — No 4 Fraction (lb/cu. ft.)	116	117	124	109
Absorption, + No. 4 Fraction (%)	4 0	5 6	3 6	4.7

TABLE 2

SUMMARY OF TEST DATA, SOIL 10

		CBR					W & D		F & T		Comp. Str. (psi)	
		After Cure[a]	After Immersion (days)				Cycles Completed	Loss in weight	Cycles Completed	Loss in weight	After Cure	After F&T
Admixture	%		2	7	14	28		%		%		
None		44	50				0	—	2(max.)	10	61(avg.)	0-10
(1)Portland Cement	1	95					12	15	7	41	179	15
	2	138		148	156	136	12	8	12	52	235	—
	4	225					12	3	12	11	263	402
(2)Lime	1	63					12	14	2	39	68	—
	2	66		36	42	61	12	11	2	23	52	—
	4	70					12	9	0	—	60	41
(3)Lime-flyash	1	60					0	—	2	16	89	—
	2	59		61	61	50	0	—	1	7	—	—
	4	65					0	—	1	11	112	5
(4)Calcium Acrylate	1						0	—	1	8	58	25
	2						2	13	4	12	113	—
	4						4	10	12	10	204	76
(5)Emulsified Asphalt	1/2	135	43				0	—	12	41	208	10
	1	135	33				12	4	12	8	201	47
	2	151	18				12	4	12	2	226	42
(6)Cutback Asph. (RC-2)	1/2	140	75				12	8	8	5	163	15
	1	100	76				12	5	12	6	257	17
	2	85	75				12	6	12	2	178	66
(7)Road Tar (RT-5)	1/2	130	68				12	10	7	43	158	—
	1	112	63				12	11	9	56	139	62
	2	98	70				12	16	12	25	106	12

[a] Cure for no admixture - 5 days at 73 F. and 100% RH
Cure for admixtures 1, 2, and 3 - 5 days at 73 F. and 100% RH plus 2 days immersed
Cure for admixtures 5, 6, and 7 - Storage at 140 F. to constant weight

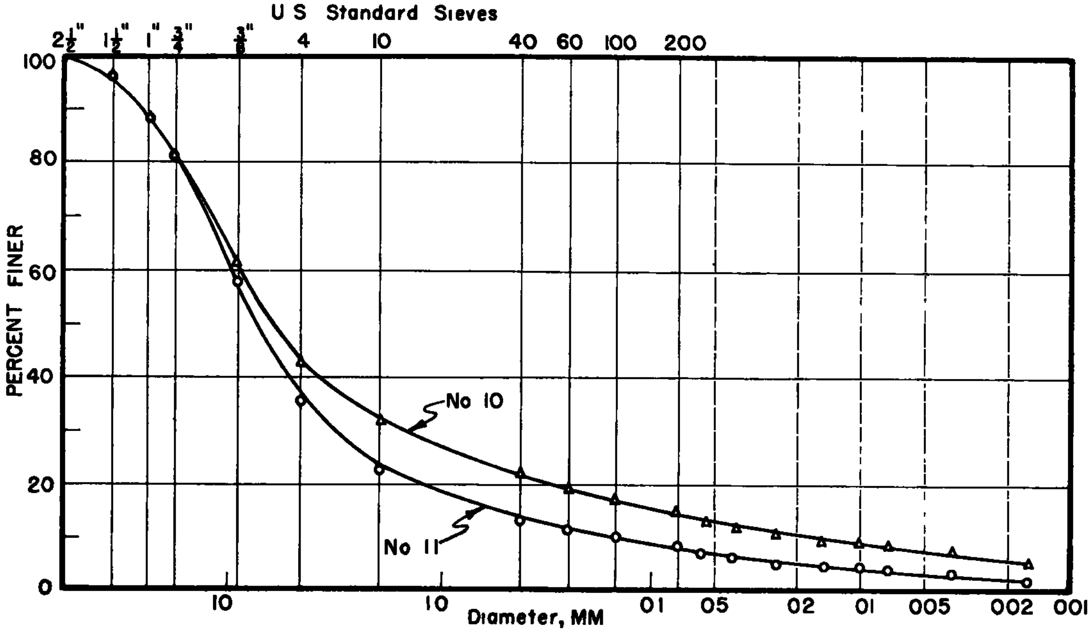

Figure 1. Grain-size distribution for Soils 10 and 11.

to determine the efficiency of the various admixtures when combined with the four soils. During the early stages of the investigation these included the California Bearing Ratio test, wetting-and-drying and freezing-and-thawing tests, and unconfined-compressive-strength tests. CBR tests were performed at the end of a suitable curing period for each specimen and after various lengths of immersion in water. Wetting-and-drying and freezing-and-thawing tests were performed in accordance with ASTM and AASHO specifications for soil-cement testing. Unconfined-compressive-strength tests were made on Proctor specimens at the end of the curing period and after wetting and drying or freezing and thawing. These later tests were made

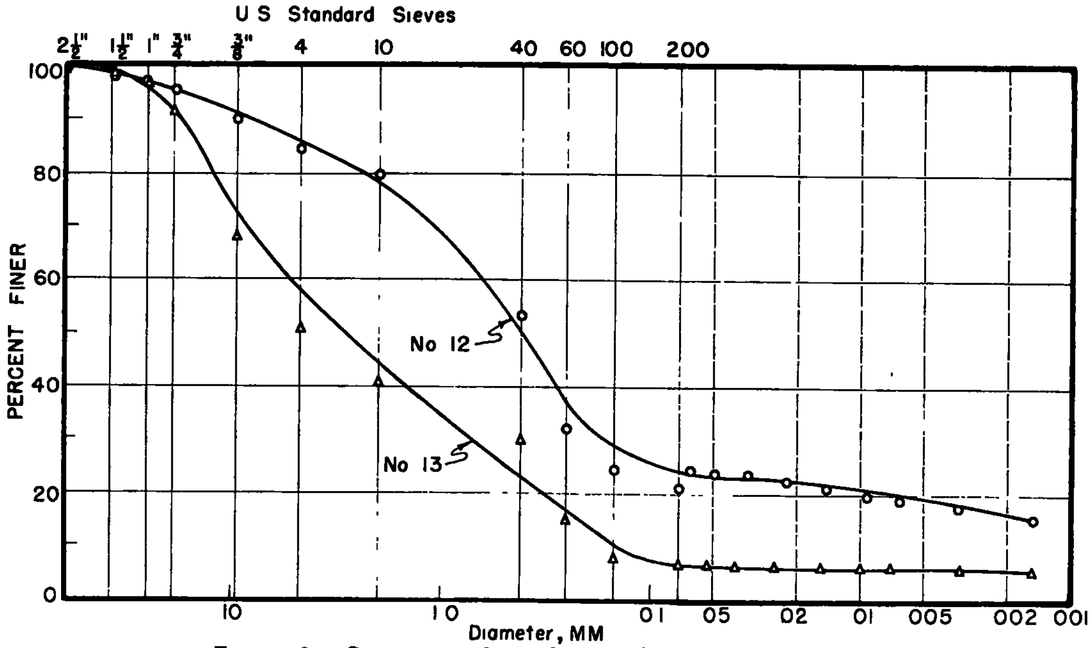

Figure 2. Grain-size distribution for Soils 12 and 13.

TABLE 3
SUMMARY OF TEST DATA, SOIL 11

Admixture	%	W & D Cycles Completed	W & D Loss in weight, %	F & T Cycles Completed	F & T Loss in weight, %	Comp. Str. (psi) After Cure	Comp. Str. (psi) After F & T
		0	–	0	–	21(avg.)	14(avg.)
(1)Portland Cement	1	12	8	12	66	63	–
	2	12	6	12	12	140	–
	4	12	3	12	3	252	388
(2)Lime	1	12	19	1	14	37	8
	2	12	11	1	10	17	11
	4	12	8	1	20	27	–
(3)Emulsified Asphalt	1/2	12	16	12	30	185	–
	1	12	13	12	36	194	63
	2	12	8	12	10	232	103
(4)Cutback Asphalt (RC-2)	1/2	12	29	7	45	284	–
	1	12	30	12	19	205	–
	2	12	15	12	10	143	86
(5)Road Tar (RT-5)	1/2	7	40	6	36	136	–
	1	6	38	6	48	122	–
	2	6	29	12	38	56	37

either after the specimen had undergone a full 12 cycles of the weathering test or at the end of the cycle in which a companion brushed specimen was observed to fail.

The results of these strength tests on specimens at the end of wetting and drying are not conclusive, in as much as the wetting-and-drying cycle ends with the specimens in an over-dried condition. In this condition all specimens, regardless of type or amount of admixture, showed quite high strengths. The similar tests performed at the end of a cycle of freezing and thawing, were quite informative, however, since the specimen in a thawed condition had a high moisture content and represented

TABLE 4
SUMMARY OF TEST DATA, SOIL 12

Admixture	%	CBR After Cure[a]	CBR After Immersion (days) 2	7	14	28	W & D Cycles Completed	W & D Loss in weight %	F & T Cycles Completed	F & T Loss in weight %	Comp. Str. (psi) After Cure	Comp. Str. (psi) After F & T
None							0	–	3(max.)	2	20(avg.)	0-6
(1)Portland Cement	1	143					12	40	12	51	58	21
	2	181		270	232	230	12	18	12	8	125	81
	4	377					12	5	12	4	216	258
(2)Lime	1	75					2	9	6	48	37	2
	2	68		69	81	118	11	9	9	32	47	–
	4	68					5	2	12	46	55	–
(3)Lime-flyash	1						0	–	1	2	24	7
	2						3	59	2	5	–	7
	4						7	83	12	54	65	10
(4)Calcium Acrylate	1						0	–	2	21	20	6
	2						2	6	12	18	46	11
	4						1	2	12	6	75	51
(5)Emulsified Asphalt	1/2	86	27				0	–	3	8	92	–
	1	69	21				3	4	4	21	122	–
	2	42	18				12	27	12	35	93	31
(6)Cutback Asphalt (RC-2)	1/2	80	57				6	18	5	17	68	–
	1	48	57				1	7	8	43	76	–
	2	28	18				2	1	12	20	65	10
(7)Road Tar (RT-5)	1/2	77	28				0	–	1	26	88	–
	1	56	15				6	69	8	52	103	–
	2	28	20				12	58	12	48	60	18

[a] Cure for no admixture - 5 days at 73F and 100% RH
Cure for admixtures 1 and 2 - 5 days at 73F and 100% RH plus 2 days immersed
Cure for admixtures 5, 6 and 7 - Storage at 140F to constant weight

probably the worst condition likely to occur in the field.

Results of the tests outlined above are summarized for the four soils in Tables 2, 3, 4, and 5, respectively. All admixtures except calcium chloride were included with Soils 10 and 12, while only portland cement, lime, emulsified asphalt, cutback asphalt, and road tar were added to Soils 11 and 13. Curing periods used for CBR specimens are indicated on the appropriate tables. For specimens subjected to the weathering tests, those containing portland cement, lime and flyash, or calcium acrylate were cured by storage for one week in a standard moist room at 73 F. and 100 percent relative humidity. Similar specimens containing emulsified or cutback asphalt or road tar were cured at 140 F. until they reached constant weight.

Examination of the four tables referred to above leads to the following observations:

1. All four soils, when stabilized with 4 percent portland cement by weight and moist cured for one week, develop high values of CBR which are not detrimentally affected by prolonged immersion, strongly resist the action of wetting and drying or freezing and thawing, and develop high unconfined compressive strengths which are not reduced by weathering of 12 cycles duration.

2. Emulsified and cutback (RC-2) asphalts are effective in reducing damage caused by weathering, comparable in this respect when used in quantities of 2 percent by weight with portland-cement stabilization. Compressive strengths of specimens stabilized with these materials, however, are considerably reduced after freezing and thawing, as are CBR values after immersion for relatively short periods. In many cases, however, the reduced values after immersion or freezing and thawing are greater than the original values of the soil alone.

3. Hydrated lime, lime and flyash, calcium acrylate and road tar appear to be inadequate as stabilizing agents for these soils. Limited exceptions may be noted from the tables.

The use of calcium chloride in this study was limited to its inclusion in specimens subjected to both types of weathering tests. It was used in quantities of 1/2, 1, 2, and 4 percent by weight and cured either by one week storage in a standard moist room or one week storage in air in the laboratory where the specimens were originally prepared. All specimens stabilized with calcium chloride acted very much as did the same soil when tested without stabilizing admixtures. Specimens subjected to wetting and drying failed immediately upon the first immersion, and those subjected to freezing and thawing fell apart during the first or second cycle. These results indicate that calcium chloride does not provide any stabilizing action of the nature sought in this investigation. This does not imply that calcium chloride, used as a construction aid, may not result in such improvements as greater and more-uniform densities which will be of lasting benefit to the base.

TABLE 5
SUMMARY OF TEST DATA, SOIL 13

		W & D		F & T		Comp. Str. (psi)	
Admixtures	%	Cycles Completed	Loss in weight, %	Cycles Completed	Loss in weight, %	After Cure	After F & T
None		0	–	0	–	3(avg.)	14(avg.)
(1)Portland Cement	1	12	19	12	19	75	39
	2	12	7	12	6	167	150
	4	12	3	12	2	335	348
(2)Lime	1	1	4	1	18	12	4
	2	12	44	1	19	17	–
	4	12	27	1	9	23	2
(3)Emulsified Asphalt	1/2	10	38	12	58	41	10
	1	12	39	12	46	51	14
	2	12	22	12	14	137	18
(4)Cutback Asphalt (RC-2)	1/2	12	44	11	44	58	–
	1	12	44	12	19	78	44
	2	12	36	12	13	130	56
(5)Road Tar (RT-5)	1/2	2	28	1	21	22	20
	1	2	39	1	22	11	4
	2	2	40	8	56	17	7

Perhaps the most-notable result outlined above is the marked increase in durability caused by the addition of cement to all of the soils tested. In an effort to determine whether this would apply to poorer materials, small samples of three additional cherts were obtained. The gradation and clay content of these materials was such that they would not be at all acceptable under present base specifications. Each of the three was stabilized by addition of 4 percent by weight of portland cement and subjected to both wetting-and-drying and freezing-and-thawing tests. Specimens of all three survived the 12 cycles of wetting and drying with weight losses of less than 12 percent. Similar specimens also survived 12 cycles of freezing and thawing. In two cases the losses in weight were less than 15 percent and in the third the loss was approximately 35 percent.

In some areas where cherts and gravels are largely nonexistent, or are of very inferior quality, considerable deposits of sand may be found. It was felt that some advantage, either in performance or economy or both, might accrue from blending these sands with such cherts and gravels as might be obtained. To check this hypothesis samples of two sands were obtained. One was a very fine sand from Madison County and the other a medium coarse concrete sand obtained from Knoxville. Specimens containing combinations of these sands with Soils 10 and 11, the two cherts used, throughout the study, were subjected to weathering tests. Results are shown in Table 6. The sands were designated as Soils 17 and 18, respectively. All specimens containing combinations of sand and chert were stabilized with portland cement.

Comparison of Table 5 and Tables 2 and 3 shows that additions of as much as 20 percent sand to either of the cherts appears to have little influence upon the durability of the materials when stabilized with portland cement. In some cases the accumulative weight losses are slightly greater and in others slightly less when the sand is included.

There is some reason to believe that none of the tests reported thus far are entirely satisfactory for determining the suitability of such materials as those studied in this project for use as base-course materials. As the size of the individual particles in the soil increases, the effect of the restraint of the steel wall of the CBR mold becomes ever more important. At the same time, the ratio of the size of the piston forced into the specimen to the size of an individual particle becomes smaller and the possibility of any one particular particle unduly influencing the test results becomes greater.

The durability tests outlined above appear to have merit in establishing the

TABLE 6
SUMMARY OF DURABILITY TESTS, COMBINATIONS OF CHERTS AND SANDS

Soil	Portland Cement %	Wetting and Drying: Cycles Completed before Failure	Wetting and Drying: Loss in Weight During Indicated Cycles (%)	Freezing and Thawing: Cycles Completed before Failure	Freezing and Thawing: Loss in Weight During Indicated Cycles (%)
90% No. 10, 10% No. 17	1	12	22	7	52
	4	12	4	12	4
80% No. 10, 20% No. 17	1	12	37	12	65
	4	12	4	12	5
90% No. 11, 10% No. 17	0	0	–	0	–
	1	10	40	10	47
	2	12	15	12	20
	4	12	5	12	5
80% No. 11, 20% No. 17	0	0	–	0	–
	1	12	35	11	50
	2	12	10	12	14
	4	12	3	12	1
80% No. 11, 20% No. 18	0	0	–	0	–
	1	12	38	3	46
	2	12	14	12	30
	4	12	3	12	6
70% No. 11, 30% No. 18	0	0	–	0	–
	1	0	–	3	30
	2	12	16	12	32
	4	12	8	12	3

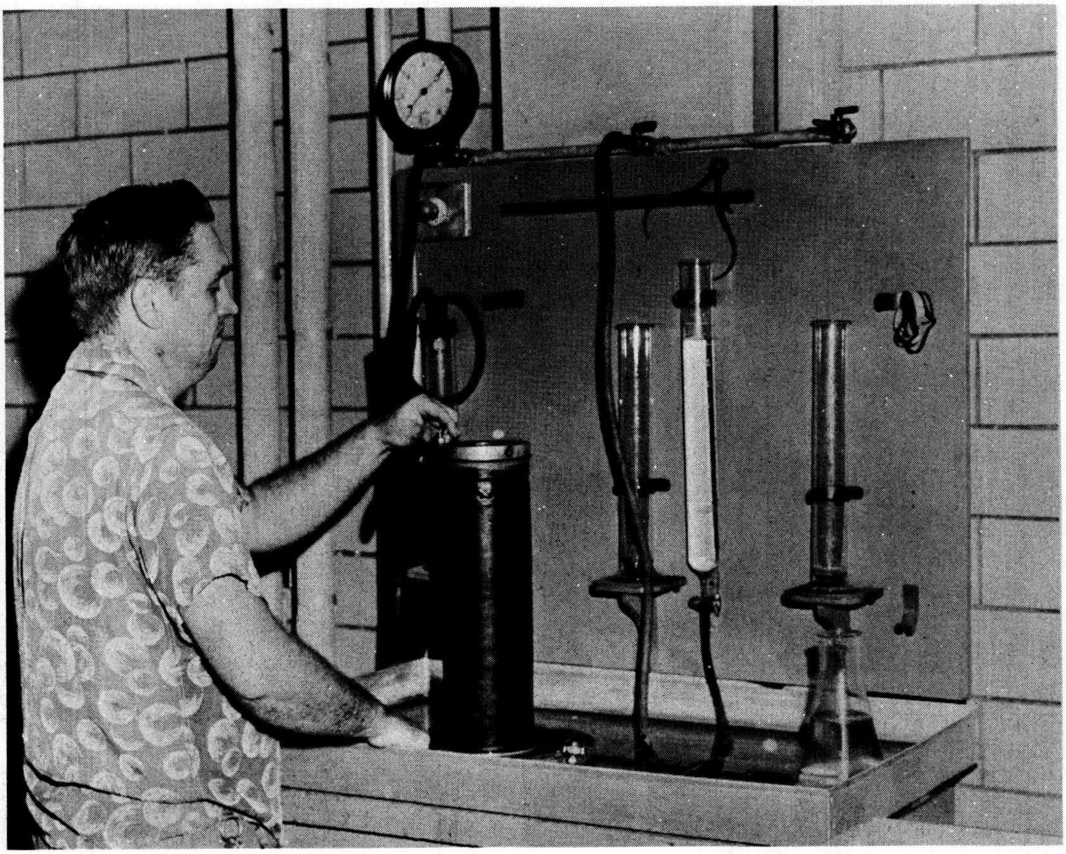

Figure 3. Saturating specimen for triaxial testing.

relative durability of the different soils or stabilized mixes. It may be argued, however, that they are unduly severe for materials used in this climate, since those having known good performance records failed rapidly under either type of exposure. Unconfined - compressive - strength tests made at the end of the curing periods appear to give some information concerning the relative efficiency of admixtures. Unfortunately, since all admixtures do not require the same type of curing conditions, specimens are not in comparable conditions at the time of test, some having a high moisture content and others being oven dry. In an effort to overcome these objections, a test was sought which would measure some basic property of the soil which in turn might indicate its suitability for base use.

The use of a triaxial-compression test was immediately suggested, since results of such tests may be interpreted to give the angle of internal friction and the cohesion of the material, these being the properties which are frequently assumed to control its load-carrying ability. Unfortunately, most existing commercially available triaxial machines were built for testing fine-grained materials such as sands, silts, and clays. The size of the specimen is usually quite small. Since this study was concerned with cherts and gravels, it was felt highly desirable that fractions at least as big as 3/4 inch be included in the test specimen.

After some investigation it was found that the Kansas State Highway Department had built and was using a device suitable for testing granular materials in triaxial compression. A set of drawings for this device was provided by that organization and the instrument was built in the shop of the engineering experiment station at the university.

Specimens used in this test are 5 inches in diameter and 14 inches high. They are molded in nine layers in a split mold, with

Figure 4. Testing specimen in triaxial-compression machine.

each layer receiving 39 blows from a standard Proctor hammer. The restraint bands holding the mold together are then removed, the mold separated, and the specimen lifted out. After suitable curing it is ready to be tested.

The triaxial-compression test chamber consists of four major components, a base plate, a specimen head, a retaining wall, and a chamber head. The chamber head is fitted with a bronze sleeve through which a load piston may be passed.

The specimen to be tested is placed on a pedestal in the center of the base plate. The top of this pedestal contains a porous stone disc having essentially the same diameter as the specimen, i. e., 5 inches. A neoprene rubber sleeve with an inside diameter of 5 inches and a wall thickness of 0. 03 inch is then worked down over the specimen. This sleeve, 17 inches long, extends down over the base pedestal and up beyond the top of the specimen. The specimen head, containing another porous stone disc, is slipped into the upper end of the sleeve until it rests upon the top of the specimen. Rubber bands are used to keep the sleeve pressed tightly against both the base pedestal and the specimen head.

Except under unusual conditions, all specimens are saturated prior to test. A vacuum is applied to the cylinder head, drawing the air from the specimen and sucking the sleeve against its sides. Water is then admitted to the specimen through the porous stone disc in the base pedestal. This process is shown in Figure 3. The process is continued until 200 cc. of water have been drawn through the specimen. At that point, both the vacuum and water supply valves are closed simultaneously. The amount of water taken by the specimen and the amount of time required by this process are recorded.

The retaining wall, either a seamless steel or a plexiglass tube having an inside diameter of 8 inches, is lowered over the specimen and seated against a gasket in-set in the base plate. The chamber head is seated on top of the retaining wall and bolts connecting it with the base plate securely tightened. The entire assembly, containing the saturated specimen, is lifted into a compression machine and the load piston is lowered through the chamber head until it rests on the specimen head. The moving head of the compression machine is brought into contact with the load piston.

Compressed air at a regulated pressure of 10, 20, 30, or 40 psi. is applied to a reservoir of water, forcing the water into the test chamber where it surrounds the specimen, applying a uniform lateral pressure. A partially filled burette is connected to the valve through which the specimen was saturated and the valve is opened. Any change in volume of the specimen will be reflected in the level of the water in the burette. The specimen is then ready to be tested.

Load readings and readings of water level in the burette are recorded every 30 seconds. The tests are continued until the specimen has failed, as is indicated by progressive falling off of load. Figure 4 shows such a test in progress.

Four similar specimens are made from each soil or each combination of soil and stabilizing admixture. One is tested at each of the lateral pressures indicated above. In the earlier tests only three specimens were made and tested at lateral pressures of 10, 20 and 30 psi. From the data obtained, Mohr's circles of stress may be constructed and the angle of internal friction and cohesion of the material determined.

All four of the basic soils and combinations of these soils with sand were tested as outlined above with several admixtures. Results of these tests are summarized in Table 7. Examination of these data leads to the following observations:

1. The addition of 4 percent portland cement to any of the soils generally increased their angle of internal friction from 9 to 14 deg. and from 30 to 45 psi. Soil 13 is the only exception, with the angle of internal friction increased by only 2 deg. but the cohesion by 66 psi.

2. When specimens stabilized with asphalt are tested in triaxial compression, the asphalt appears to act as a lubricant, generally increasing the cohesion of the material but decreasing its angle of internal friction.

3. The blending of 20 percent of sand with either of the cherts appears to slightly improve the angle of internal friction when stabilized with 4 percent of cement. The cohesion is slightly decreased in some cases. The use of the sand is highly beneficial to Soil 10 when stabilized with emulsified asphalt, largely offsetting the decrease in the angle of internal friction due to the addition of 2 percent asphalt alone to Soil 10.

4. The effect of blending sand with other base materials appears to vary considerably from one material to another and from one admixture to another for any given base material. This suggests that any such blending should be carefully considered and tests performed on the actual materials involved prior to acceptance of the practice.

TABLE 7
SUMMARY OF ALL TRIAXIAL TEST DATA
(All specimens cured 1 week and saturated prior to test)

	Type Admixture	%	Average angle of internal friction (deg)	Average cohesion (psi)
No. 10	None		43.5	9
	Cement	4	57	40
	Em. asph.	2	38	43
80% No. 10, 20% No. 18	None		44	14
	Cement	4	58	53.5
	Em. asph.	2	43	36
No. 11	None		41.5	13.5
	Cement	4	55.5	57.5
	Em. asph.	2	36	53
	RC-2	2	42.5	25.5
	RT-5	5	45	14
80% No. 11, 20% No. 18	Cement	4	57.5	47.5
	Em. asph.	2	54	13
	RC-2	2	39	26.5
No. 12	None		36.5	4.5
	Cement	4	51	38
	Em. asph.	2	32	9
	RC-2	2	33	1
80% No. 12, 20% No. 18	None		38	10
	Cement	4	49	42.5
	RC-2	2	29	6
No. 13	None		41	6
	Cement	4	43	72.5
	RC-2	2	34	9
80% No. 13, 20% No. 18	Cement	4	49.5	44
	RC-2	2	34	6.5

A careful analysis of the data presented in the foregoing shows that the various tests do not always rate the soils in the same order in determining their suitability for base use, as should be the case if all tests were equally valid in evaluating the properties of the various materials. For example, CBR tests performed on Soils 10, 11, and 12 without stabilizing admixtures indicated their usefulness, in decreasing order, to be Nos. 11, 10, and 12. Un-

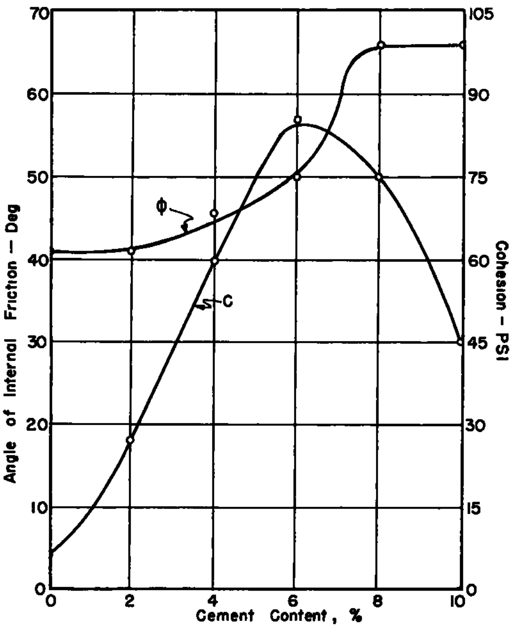

Figure 5. Effect of cement content on triaxial-test results.

confined compression tests performed on the same materials, and also on Soil 13, indicated the value of soils in decreasing order to be Nos. 10, 11, 12, and 13. Triaxial compression tests showed Soil 10 and 11 to be quite similar, followed in decreasing order by Soil 13 and Soil 12. Since specimens made from all soils failed immediately in the wetting-and-drying test and during early cycles in the freezing-and-thawing tests, it is felt that these tests do not effectively differentiate between the materials tested.

When the four basic materials were stabilized with 4 percent of portland cement similar differences between tests were observed. CBR tests on Soils 10, 11, and 12 at the end of the curing period indicated their value in decreasing order to be Soils 12, 11, and 10. Specimens made from all soils performed very well in both wetting-and-drying and freezing-and-thawing tests. When both tests are taken into account, what slight preference exists would indicate the relative value of the soils to be Nos. 13, 11, 12, and 10.

Unconfined-compressive-strength tests at the end of the curing period rated the soils as Nos. 13, 10, 11, and 12, while similar tests after 12 cycles of freezing and thawing showed their relative values to be Nos. 10, 11, 13, and 12. On the basis of the angle of internal friction, as determined from triaxial-compression tests, the soils would be rated Nos. 10, 11, 12, and 13, while on the basis of cohesion they would be rated Nos. 13, 11, 10, and 12. In using the triaxial-compression test for evaluation of such materials, however, it seems proper that both the angle of internal friction and the cohesion should be taken into account. If this is done the soils would probably be rated Nos. 11, 10, 13, and 12.

The tests appear to be in greater conformity with respect to indicating the relative effects of admixtures upon the soils that in differentiating between the soils themselves. For example, CBR tests on Soil 10 indicated the addition of 1/2 percent of emulsified asphalt to be more beneficial than the addition of 1 percent of portland cement, 1 percent of emulsified asphalt to be slightly superior to 2 percent of portland cement, but 2 percent of asphalt to be considerably inferior to 4 percent of portland cement. Results of unconfined-compressive-strength tests on similar combinations showed 1/2 percent of emulsified asphalt to be superior to 1 percent of portland cement, 1 percent of emulsified asphalt to be slightly inferior to 2 percent of portland cement, but 2 percent of asphalt to be notably inferior to 4 percent of portland cement. Results of durability tests on Soil 10 indicate a similar order.

The most-difficult of the tests to perform is the triaxial-compression test. Greater care is required in the preparation of the specimen and in its testing. It is believed, however, that this test is highly representative of conditions existing in a highway base and that, providing information concerning two of the fundamental properties of the base material, its angle of internal friction and its cohesion, it very probably offers more usable information concerning the suitability of the material than do any of the other tests.

Toward the end of the testing program, a limited series of triaxial-compression tests was performed to investigate the effect of percentages of admixture varying over a wider range than that included through the major portion of the study. The results of these tests were extremely interesting and perhaps indicate the possibility of using a technique differing from those employed in the past in determining the optimum percentage of an admixture for use in stabilization.

In the first series of tests, one of the gravels was stabilized with portland cement in quantities of 2, 4, 6, 8 and 10 percent by weight. Each combination was tested in triaxial compression and its angle of internal friction and cohesion determined. The values have been plotted in Figure 5. Lateral pressures of 20, 40, and 60 psi. were used in these tests. It may be observed that the cohesion of the mixture increased rapidly to a maximum at a cement content of 6 percent and decreased beyond that point, apparently indicating that the material had become brittle. The angle of internal friction increased more slowly at low cement factors and appeared to reach a maximum at 8 percent cement content, showing no increase when the cement factor was raised to 10 percent. In supplemental tests where lateral pressures were limited to the 10-, 20-, 30- and 40-psi. levels used in the remainder of the study, the maximum value of angle of friction was reached at, or slightly beyond, 6 percent of cement. This appears to indicate that the stabilized gravel had reached a strength at which the material was no longer influenced by the restraining pressures chosen.

It is generally agreed that road or base materials, with the exception of very rigid ones, such as portland-cement concrete, gain much of their load-supporting-and-distributing ability from the restraint exercised on material immediately beneath the loaded area by adjacent material. It was demonstrated in the early phases of the study that a cylinder of this soil alone, or even in combination with cement, had relatively low ultimate strength when tested in unconfined compression. Under the influence of lateral restraint, however, these ultimate strengths were much increased. The data shown in Figure 5 indicate that if the lateral restraint which a material is capable of developing is known, or can be satisfactorily determined, triaxial tests such as those described above with lateral pressures limited in magnitude to the estimated restraint on the material in place will give a clear indication of the maximum percentage of admixture which may be beneficially employed. In the case of the gravel used in these tests, if the maximum lateral pressure which may be developed in such a base is assumed to be 60 psi., Figure 5 clearly demonstrates that little can be obtained by the use of cement contents in excess of 8 percent. In view of the performance of the cohesion, it seems likely that a cement content in the order of 6 percent by weight would be more nearly optimum.

A similar series of tests were performed on the same material using a cutback asphalt at residual asphalt contents of 1, 2, and 4 percent. Results shown in Figure 6 indicate that the asphalt was acting somewhat as a lubricant with the cohesion of the mixture increasing and the angle of internal friction decreasing as the percentage of asphalt was increased.

Although this project was undertaken with the hope of developing techniques for the construction of low-cost roads from local base materials, it is felt that the results may have considerable significance with respect to construction and reconstruction on the primary road system. With vehicle weights continually increasing, and no apparent end to this trend in sight, it becomes evident that our primary highways will have to be built to standards not previously considered. In many areas, aside from economic considerations, the availability of materials definitely limits the extent to which the thickness of base courses can be increased. The notable improvement in the qualities of the materials included in this study which is brought about by the addition of small percentages of some of the admixtures suggests that the use of such stabilization practices might become a standard prac-

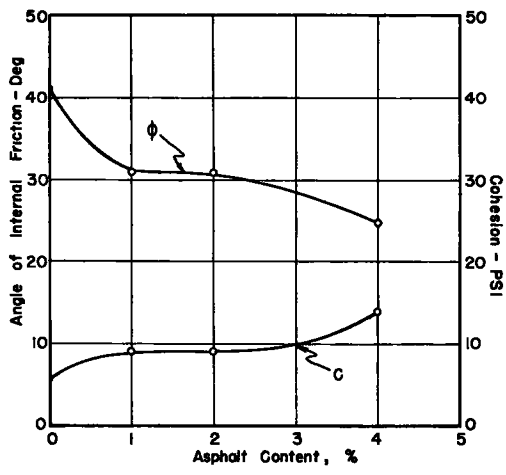

Figure 6. Effect of asphalt content on triaxial-test results.

tice in primary road construction. If the material can be batched and mixed at a central plant, before application to the road bed, there is every reason to believe that small percentages of admixture may be economically and beneficially employed.

It is appreciated that the data reported have all resulted from laboratory investigations. Although it is believed that the angle of internal friction and the cohesion of a material are good measures of that material's load-bearing ability and that changes in these factors may appropriately be used to determine changes in the load-carrying capacity of the material, it must be acknowledged that data presently available are not sufficient to indicate what magnitude of angle of internal friction and cohesion are required in any specific instance. Such data must be collected through actual field experimentation. In this regard, it is anticipated that the Tennessee Department of Highways and Public Works will construct several sections of stabilized base in the near future for the purpose of providing correlation between results of laboratory investigations and actual performance under traffic in the field. In this way it is hoped that a suitable method of highway base design may be evolved.

Discussion

RICHARD H. MILLER, Assistant Professor of Civil Engineering, Clarkson College of Technology — The use of lime and flyash shows very-low values of compressive strength and low resistance to cycles of freezing and thawing. The curing period for these samples consisted of 5 days at 73 F. and 100 percent relative humidity plus 2 days water immersion. It is widely recognized that compositions employing lime do not set up as do hydraulic cements but respond to cycles of high-low humidity as well as presence of CO_2, which slowly forms a recrystallized hydrate and calcium carbonate matrix. It is felt that the tests as presented in this paper might not be indicative of results obtained in field operations using materials such as described in this paper. The curing of the compositions are brought about under entirely different conditions.

The lime-and-flyash mixtures utilize the lime-hardening reaction but, in addition, would be expected to produce a hydraulic set as a result of the pozzolanic reaction between the lime and flyash. This reaction, however, would proceed slowly; if a 7-day cure is contemplated in laboratory testing, it would be necessary to accelerate the reactions, preferably by the use of higher temperatures during the curing period.

This subject has been reported upon in earlier papers[1] presented to the Highway Research Board and reference is also made to Whitehurst's previous paper "Durability Tests on Lime-Stabilized Soils".[2]

Tables of the subject paper show percentages of lime and flyash used of 1, 2, and 4 percent. It is not stated whether these percentages are of lime, flyash, or of a combination of lime and flyash. If the percentages as stated are the total percentages of the combination of lime and flyash used in the mixtures, as they appear to be; then the amounts are far too small and it is not surprising that the results were poor. It is necessary generally to add to the soil 5 percent of lime and 10 to 15 percent of flyash in order to derive a beneficial stabilizing effect. Previous investigations have shown that both the amount of lime and the amount of flyash must be carefully controlled. In particular, the use of too much flyash may actually

TABLE A
PROPERTIES OF HYDRATED LIME AND FLYASH

Chemical Analysis	Hydrated Lime	Flyash
SiO_2	1.0	40.32
Fe_2O_3	0.4	13.39
FeO	0.0	3.95
Al_2O_3	0.2	32.92
CaO	47.8	2.34
MgO	33.8	0.74
Loss on Ignition	16.3	5.79
CO_2	0.8	–
H_2O	0.5	–
Sieve Analysis		
Sieve No.		
60 (total percent retained)	1.0	2.0
100 " " "	2.8	10.1
200 " " "	5.6	21.0
Specific Gravity	2.60	2.20
Dry Rodded Density - lb. per cu. ft.	45	60

[1] Highway Research Board Proceedings, V. 30, 1950, pp. 489-502, and V. 31, 1952, pp. 511-528, and Bulletin 69, pp 1-28.

[2] Highway Research Board Proceedings, V. 31, pp. 529-540.

produce a mixture which is poorer in some properties than the original soil.

It is not clear, in the paper, that lime and flyash are two distinctly separate materials. The paper states, referring to lime and flyash: "The former is a waste material of lime production." Actually hydrated lime is the major product of the lime industry, while flyash is a fine ash collected when pulverized coal is burned. A detailed analysis of each material is shown in Table A. It would also be helpful, to one not familiar with Tennessee gravel or chert, in evaluating the study, to have available the grain-size curves and AASHO classification of the soils used.

E.A. WHITEHURST, Closure — With reference to Miller's discussion, the author appears to have erred in not more clearly defining the material referred to in the paper as "lime and flyash". It was not intended that this material be identified as a combination of hydrated lime and flyash, with both of which materials the author is generally familiar. The material is, in the words of the company representative who provided it for use in this study, a byproduct and has the chemical characteristics, as reported by the company, shown in Table B.

It is agreed that the compressive strength of soils stabilized with combinations of hydrated lime and flyash are generally appreciably increased by subjection to prolonged curing. It was the purpose of this study, however, to devise means not only of stabilizing bases for highways in the state highway system, but also roads in the county road system, many of which may not receive the degree of maintenance which is desirable. It was felt, therefore, that the study should be limited to the determination of the effectiveness of various potential stabilizing admixtures after relatively short curing periods.

TABLE B
CHEMICAL PROPERTIES OF LIME-FLYASH MIXTURE

	%
$Ca(OH)_2$	47.36
$CaCO_3$	25.98
CaO(x)	3.22
SiO_2	8.18
Al_2O_3	0.97
Fe_2O_3	0.42
MgO	0.52
C	13.35

Note: The CaO(x) refers to calcia that is bound to silica and alumina, and which is, hence, unavailable.

Although the material under discussion is a byproduct and would probably be relatively inexpensive, no sources are available in or near those areas of Tennessee in which poor base materials are prevalent. Since long hauls would be involved, the use of percentages such as those suggested by Miller would be most uneconomical. The quantities chosen, 1, 2, and 4 percent by weight, were believed to be practical for field use as well as comparable, quantitatively, to the other admixtures investigated.

The physical properties of the soils investigated, together with their grain-size distributions have been provided in the paper as revised for printing.