

Effect of Chemicals on Soil-Cement Stabilization

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The effects of certain chemicals on soil-cement mixtures were quantitatively measured by three tests: unconfined compressive strength, triaxial compressive strength, and Iowa freeze-thaw and wet-dry durability tests. The chemicals were compounds of sodium, calcium and magnesium, and also commercial lime. The eight soils used varied widely in properties due to differences in sampling location, depth, texture, and type of clay mineral.

The 7-day and 28-day cured, 1-day immersed, unconfined compressive strengths of specimens compacted to near standard Proctor density at optimum moisture content indicated that organic top soils benefited from the incorporation of sulfates when the soils were acidic and low in clay content. With increasing clay content and an alkaline environment, the addition of calcium and magnesium ions generally gave high strengths. The B and C horizon clay soils containing cement seemed to respond very favorably to additives of lime, sodium hydroxide, or sodium carbonate, the latter only with soils having near neutral pH.

Stabilized soil mixes that attained a 7-day unconfined compressive strength of 250 psi or more were further evaluated in the Iowa freeze-thaw and wet-dry durability tests. Results verified the strength beneficiation derived from adding the chemicals to soil-cement mixtures. Also, they provided data that suggested the establishment of a functional relationship between the 14-day unconfined compressive strength and the strength at the end of 10 cycles of freeze-thaw.

The triaxial compressive strength test on the natural soil, selected soil-cement, and soil-cement-chemical mixes indicated that for the sandy soil-cement the promising chemicals containing magnesium or sodium ions increased both the cohesion and angle of internal friction. For the clay soil-cement, however, the promising chemicals containing calcium or sodium ions increased the cohesion substantially but decreased the angle of internal friction slightly.

•THE EXPANSION of road nets, with the attendant obligation to use in-place soils with low load-carrying capacity, has forced engineers to look for methods or processes to enhance the effectiveness of cement. Recent research (2, 4, 7, 8, 10, 14) has shown that the incorporation of chemicals or lime in small amounts produces a strength gain in soil-cement. These recent advances have provided the foundation for the leading ideas of this investigation; namely, the extent of strength gain as related to the type of soil, the influence of the chemicals or lime on the cohesion and angle of internal friction of the soil-cement mass, and the effectiveness of chemicals or lime measured in terms of the durability of the stabilized soil.

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MATERIALS

Soils

The eight soils used in this investigation were so selected as to have a wide range of properties, which accrue not only from variation in sampling depth and particle size distribution, but also from differences in clay mineral type. Table 1 gives property information on the eight soils that were sampled from various parts of the country and had already been classified as difficult to stabilize or problem soils.

Cement

Type I portland cement was used throughout this study.

Hydrated Lime

Two types of commercial hydrated lime were used. One was hydrated high calcium, lime A, and the other was monohydrated dolomitic, lime B.

Chemicals

The selection of chemical compounds (Table 2) was dictated primarily by the consideration that there is a similarity between these compounds and the product resulting from the combination of cement and soil, and secondarily by the research findings of earlier studies (2, 7, 8, 10).

Preparation of Mixes

A measured amount of pulverized air-dry soil passing the No. 10 U.S. standard sieve was hand mixed with a measured amount of cement. The dry mixture was placed in the mixing bowl, some of the compaction water which had been calculated to give near standard Proctor density was added; the mixture was hand mixed again; and then it was mixed in a kitchen mixer, Hobart Model C-100, at low speed, for 1 minute. Following this, the rest of the compaction water, in which a measured amount of the chemical compound had been dissolved or dispersed, was added, hand mixed, and machine mixed for another minute. To insure uniform distribution, the bowl was scraped with a trowel, the mixture was hand mixed and finally machine mixed for an additional minute.

The amount of molding water used was that which gave maximum density and not maximum strength (Table 3) since the

TABLE 1
PROPERTIES OF SOILS

Sample Designation	Location	Parent Material	Soil		Textural Composition ^a (%)				Property				Classification Eng. J AASRO						
			Group	Series	Horizon	Sampling Depth (in.)	Gravel (>2 mm)	Sand (2-0.074 mm)	Silt (0.074-0.005 mm)	Clay (<0.005 mm)	LL ^b (%)	PL ^c (%)		PI ^d (%)	C, E, C, I (meq/100g)	Chemical Carbonates ^e (%)	Organic Matter ^f (%)		
Iowa silt clay	Wayne Co.	Leached, fine-textured Wis.-age loess derived from Wis.	Planosol	Edina	A	0-12	0.0	5.5	26.0	12.0	35	27	8	19.62	0.85	5.3	3.55	Silty clay loam	A-(6)
Iowa sand	Calhoun Co.	Wisconsin glacial deposit	Wisconsin	Webster	A	0-12	0.0	13.5	47.0	28.0	58	31	27	40.00	1.50	7.5	7.00	Clay loam	A-7-5(16)
Iowa clay	Racine Co.	Wis.-age glacial deposit	Podzolic	Plainfield	A	0-120	0.0	77.0	10.0	4.0	NP	NP	NP	6.63	0.50	7.0	1.80	Clay loam	A-3
Iowa clay	Livingston Co.	Wis.-age glacial deposit largely	Humic-Gley	Clarence (Rowe)	C	46-56	0.0	10.0	52.0	34.0	36	18	18	10.80	22.50	8.3	0.70	Clay	A-6(11)
Iowa clay	Harris Co.	Wis.-age glacial deposit largely	Grumusol	Lake Charles	C	36-144	0.0	3.0	61.0	40.0	65	18	47	27.30	16.60	8.2	0.13	Clay	A-7-6(20)
Iowa clay	Ingham Co.	Wis.-age glacial deposit	Gray-Brown Podzolic	Maple Bluff	B	12-56	0.0	14.0	57.0	35.0	43	25	16	15.06	3.30	6.9	0.65	Clay	A-7-6(12)
Iowa clay	Durham Co.	Triassic sediments	Red-Yellow Podzolic	Whitman	B	18-21	0.0	13.0	22.0	50.0	74	26	48	36.20	0.94	5.4	0.27	Clay	A-7-6(20)
Iowa sand	Shoemash Co.	Cemented gravelly till	Brown Podzolic	Score Alder-wood	-	0-120	0.0	75.0	6.0	0.0	NP	NP	NP	3.32	0.20	6.0	1.60	Sandy loam	A-3

^aTextural gradation tests performed only on soil fraction passing No. 10 sieve, ASTM Method 423-54T.

^bASTM Method D423-54T.

^cASTM Method D424-54T.

^dCation exchange capacity determined by the ammonium acetate (pH = 7) method on soil fraction less than 0.42 mm.

^eCarbon exchange capacity determined by the ammonium acetate (pH = 7) method on soil fraction less than 0.42 mm.

^fGlass electrode method using suspension of 15-g soil in 30-cc distilled water.

mM = montmorillonite.

Ch-M = chlorite-montmorillonite interlayer.

Ill = illite.

Musc-III = muscovite-illite interlayer.

Ch-I = kaolinite-illite interlayer.

Ch-I = kaolinite-illite interlayer.

TABLE 2
CHEMICALS USED

Chemical	Formula	Source
Sodium hydroxide	NaOH	— ^a
Calcium sulfate	CaSO ₄	Reagent grade
Magnesium sulfate	MgSO ₄ ·7H ₂ O	Analytical reagent
Sodium sulfate	Na ₂ SO ₄	Reagent grade
Calcium chloride	CaCl ₂	— ^a
Magnesium oxide	MgO (C-1-60)	— ^a
Sodium orthosilicate	Na ₄ SiO ₄	— ^a
Sodium carbonate	Na ₂ CO ₃	— ^a

^aSupplied by the Dow Chemical Co.

differences between the two molding water contents were very small.

Molding

When mixing was completed, the bowl was covered with a damp cloth to prevent evaporation. In a drop hammer molding apparatus, 2-in. diameter by 2-in. high specimens were molded to near standard Proctor density.

Curing

Immediately after weighing and measuring, the specimens were wrapped in waxed paper and sealed with Scotch tape to prevent loss of moisture and were cured in a chamber at 95 ± 5 percent humidity and 70 ± 5 F temperature. Two curing periods were used—7 days and 28 days.

Compressive Strength Testing

At the end of the specified curing period, the specimens were removed from the curing room, unwrapped, and immersed in distilled water for 24 hours following which they were tested to failure to determine their unconfined compressive strength.

METHODS OF EVALUATION

Preliminary Investigation

In the first phase of the investigation the 7-day and 28-day cured, 1-day immersed, unconfined compressive strengths of soil-cement specimens to which various amounts of a chemical had been added were determined. Although the 7-day strength was adopted as the criterion for successful stabilization, the 28-day strength provided not only an additional check, but also gave an indication of the strength potentiality of the mix and some measure of the rate of strength change with time. The cement contents used were 4, 8, and 12 percent; the chemical contents not more than 3 percent, the lime contents 1 and 3 percent, all based on the oven-dry weight of the soil.

From these strength data, the economically lowest optimum amount of chemical was chosen. In some mixtures, maximum strength was obtained with slightly varying amounts of chemicals for the three cement contents but the same soil. Where the variation was small, the adoption of one amount of chemical for all three cement contents with the same soil seemed justifiable.

Durability Tests

All stabilized mixtures which gave a 7-day cured, 1-day immersed strength of 250 psi or higher were further evaluated by the freeze-thaw and the wet-dry durability tests.

The procedure for the freeze-thaw test used in this work was developed in the Iowa Engineering Experiment Station Laboratory (6) to suit the climate in Iowa. (Apparatus is shown in Figs. 13, 17.) Two specimens, 2 in. in diameter by 2 in. high, were molded from the mixture to be evaluated and were moist cured for seven days. At the end of this curing period, the top of each specimen was sealed with a coat of resin-base paint (Plax) before the specimens were immersed in water for 24 hours. One specimen was removed from the water and

TABLE 3
OPTIMUM MOISTURE-MAXIMUM DRY DENSITY
VALUES OF SOIL-CEMENT MIXES

Soil Type	Optimum Moisture Content (%)	Maximum Dry Density (pcf)
Iowa silt	22.5	95.0
Iowa clay	24.8	95.1
Wisconsin sand	17.0	100.5
Illinois clay	18.0	111.4
Texas clay	22.6	102.5
Michigan clay	19.5	105.5
North Carolina clay	25.7	97.2
Washington sand	12.0	122.0

placed in the specimen holder. The assembly was placed in a vacuum flask containing sufficient distilled water, at a temperature of 34 ± 1.6 F, to immerse the bottom $\frac{1}{4}$ in. of the specimen. The vacuum flask with the specimen in it was stored for 16 hours in a refrigerator, maintained at a temperature of 20 ± 2 F. Then the flask was removed and thawed for eight hours at a temperature of 77 ± 4 F. This was one cycle of freeze-thaw; the specimen was subjected to 10 cycles. At the beginning of the cycle, at the end of the freezing period and at the end of the thawing period, the height of the specimen was measured to calculate the amount of heave. The specimen was weighed at the beginning and at the end of the freeze-thaw cycles. The amount of moisture absorbed during the test could be determined from these weights. At the completion of the freeze-thaw cycles, the specimen was removed from the vacuum flask and specimen holder and tested for unconfined compressive strength. The other specimen called the "control" specimen, was left in water through all the freeze-thaw cycles. Its unconfined compressive strength was determined at the same time as that of the first specimen. The control specimen was weighed both at the end of the moist curing period and of the immersion period. The index of resistance (in percent) to the effect of freezing R_f was calculated by

$$R_f = \frac{p_f}{p_{cf}} 100 \quad (1)$$

in which

p_f = unconfined compressive strength of freeze-thaw specimen, psi, and
 p_{cf} = unconfined compressive strength of control specimen, psi.

The preparation of two identical specimens for the wet-dry test was the same as for the freeze-thaw test; in fact, all four specimens were prepared from the same batch. One of the specimens was moist cured for 7 days, then it was immersed for 7 days. The control was moist cured for 14 days. At the end of this period, the two specimens were tested for unconfined compressive strength. The resistance to the effect of immersion was calculated from

$$R_i = \frac{p_i}{p_{ci}} 100 \quad (2)$$

in which

p_i = unconfined compressive strength of immersed specimen, psi, and
 p_{ci} = unconfined compressive strength of control specimen, psi.

On the basis of the 7-day cured, 1-day immersed unconfined compressive strength and of the resistance to freeze-thaw and wet-dry tests, it was possible to select the best cement-chemical mix for each soil for further evaluation by the standard freeze-thaw test, ASTM Designation D560-57 (1, p, 1, 182).

Triaxial Compression Tests

For each soil three different mix batches were prepared as explained previously. The first was the raw soil at its optimum moisture content for standard Proctor density; the second was a combination of soil and an amount of cement equal to that used in the third batch; the third was that combination of soil, cement, and chemical which gave the best mix based on 7-day strength and resistance to freeze-thaw.

From each batch 9 cylindrical specimens, 1.312 in. diameter by 2.816 in. high, were molded using the Harvard miniature compaction apparatus, which gives a compacted sample at approximately standard Proctor density. Samples were molded in 5 layers with 5 tamps per layer for sandy soils and 8 per layer for clayey soils.

All specimens were moist cured for 7 days in the same manner as the strength specimens.

At the end of the curing period the specimens (3 for each of the 3 lateral pressures of 10, 20 and 30 psi) were tested in a triaxial compression machine. The cohesion and angle of internal friction of any particular mix were determined graphically using the Coulomb-Mohr theory defined by

$$s = c + p \tan \phi \quad (3)$$

UNCONFINED COMPRESSIVE STRENGTH OF SOIL-CEMENT MIXTURES

Iowa Silt

The test results indicate that 4 percent cement added to Iowa silt produces low strength mixtures, whose strength cannot be increased by any of the chemicals incorporated in the mix (Fig. 1). At cement contents above 4 percent, the sulfates of sodium, calcium, and magnesium increased the 7-day strength. The 28-day strength data show the same trend. Although the sulfate data contradict the destructive effect of sulfates on concrete and clays stabilized with cement (13), they are in agreement with reported results on sandy soils and especially with the effectiveness of sodium sulfate (5). Paradoxically, no beneficiation is evident when either magnesium oxide, calcium chloride, sodium hydroxide, or limes are used. This would at first seem inconsistent with the expected

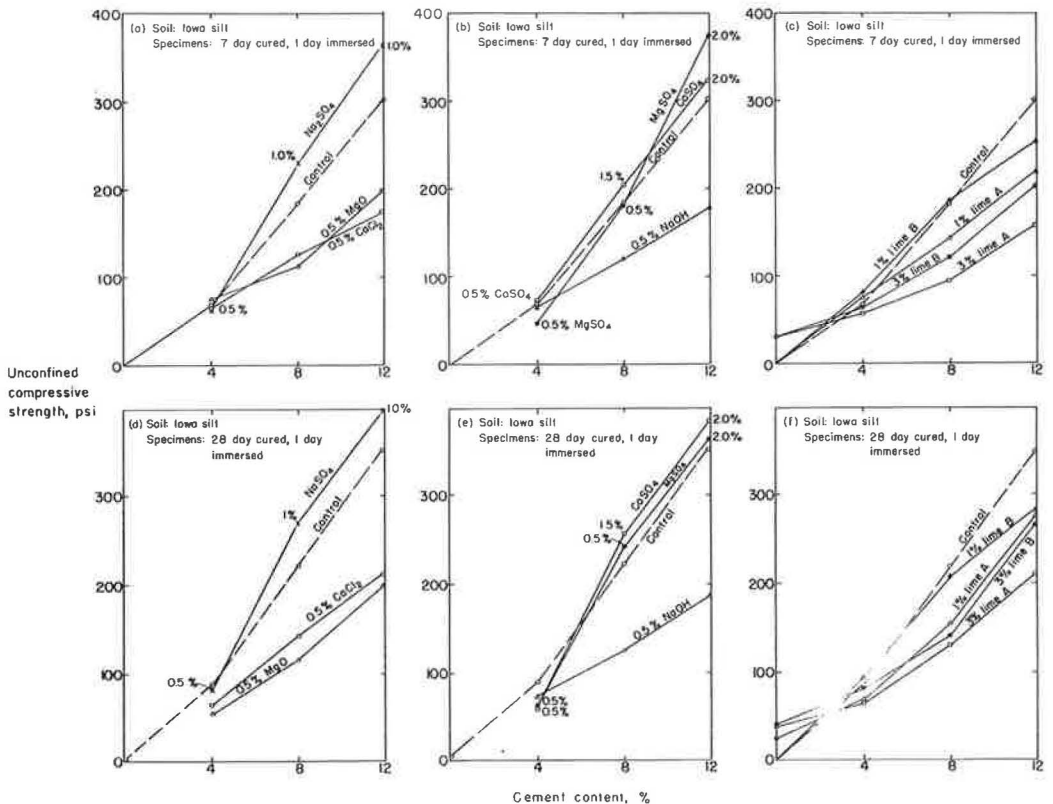


Figure 1. Unconfined compressive strengths of Iowa montmorillonitic, A horizon silt stabilized with optimum amount of chemical or hydrated lime at varying cement contents; lime A = $\text{Ca}(\text{OH})_2$, lime B = $\text{Ca}(\text{OH})_2 + \text{MgO}$.

stabilizing effect of magnesium, sodium, and calcium ions. The inconsistency may be traced to the type of compound which contains these ions and its reaction with the high organic matter content of the soil. An explanation may be based on the hypothesis that the basic compounds, such as sodium hydroxide and lime, increase the solubility of organic complexes with attendant uniform distribution through the water in the mix and interfere with the formation of silica gel. By decreasing the solubility, the sulfates reduce the interference of the organic complexes, the soil-cement reaction proceeds normally, and high strengths are attained.

A similar interpretation may be proposed for the ineffectiveness of the calcium chloride salt, which is supposed to have a catalytic effect on cement. Although the addition of calcium ions increases the rate of hydrolysis of the compounds in the clinker of cement, and thereby the transformation of the coagulated silicates and aluminates into crystalline bodies is enhanced and strength is attained, the organic complexes in Iowa silt seem to prevent this reaction. Inasmuch as the addition of 0.5 percent calcium chloride imparts relatively higher strength to the Iowa silt-cement mix than 1.5 percent, the theory that high concentrations of calcium chloride overcome the deleterious presence of the organic matter cannot be advanced.

The ineffectiveness of magnesium oxide seems to be derived from a poor solubility condition. Examination of the broken specimens after testing revealed distinct magnesium oxide agglomerations distributed throughout the specimen, an indication that magnesium oxide did not become part of the soil-cement structure.

Iowa Montmorillonite Clay

All the chemicals and limes used with mixes of Iowa clay-cement contributed to strength increase, indicating that they speeded up soil-cement reaction at all cement contents used (Fig. 2).

In comparison with the sulfates used, calcium chloride and magnesium oxide were very beneficial. This is in contrast with the results obtained with the organic Iowa silt soil. Although the two soils, Iowa silt and Iowa clay, are both top soils and highly organic, they respond differently to chemical stabilization. This difference may be due in part to their alkalinity or acidity. The Iowa silt is well leached and highly acid with a pH value of 5.3. The Iowa clay is not so well leached and is slightly alkaline, as indicated by a pH equal to 7.5. The low pH value might have caused a precipitation of a gel over the cement particles, thus delaying and possibly prohibiting the hydration of cement. The difference between the behavior of the two soils may also be explained by one's being silty and the other clayey. Although not fully elucidated for all chemicals, recent research (4) indicates that cement stabilization of friable loess does not benefit from the addition of lime; plastic loess does benefit.

Wisconsin Sand

Besides being organic, the nonplastic Wisconsin sand displays uniformity of particle size and relative absence of binder material. These properties make it very difficult to stabilize (Figs. 3 and 4). Cement contents up to 16 percent impart very little strength. Since molding and extrusion of specimens were difficult and specimens crumbled when handled immediately after compaction, it may be concluded that cement failed to give cohesive property to the soil. Except for magnesium sulfate, the other chemicals and lime in combination with cement did not contribute to the strength of the resulting mixture either. This supports the contention of the effectiveness of sulfates. The addition of sodium ions in the form of hydroxide, silicate, or carbonate was also ineffective. This leads to the conclusion that the formation of the rather stable Si-O-Na groups was prevented, possibly because the sodium ions preferentially attached themselves to the organic complexes.

The high strengths obtained with cement and magnesium sulfate merited further investigation. Combinations of 8, 12, and 16 percent cement with four different magnesium sulfate contents, the highest being 3 percent magnesium sulfate, revealed that 2 percent magnesium sulfate may be considered optimum (Fig. 4). The unique effectiveness of magnesium sulfate may be attributed to two factors: the incorporation of

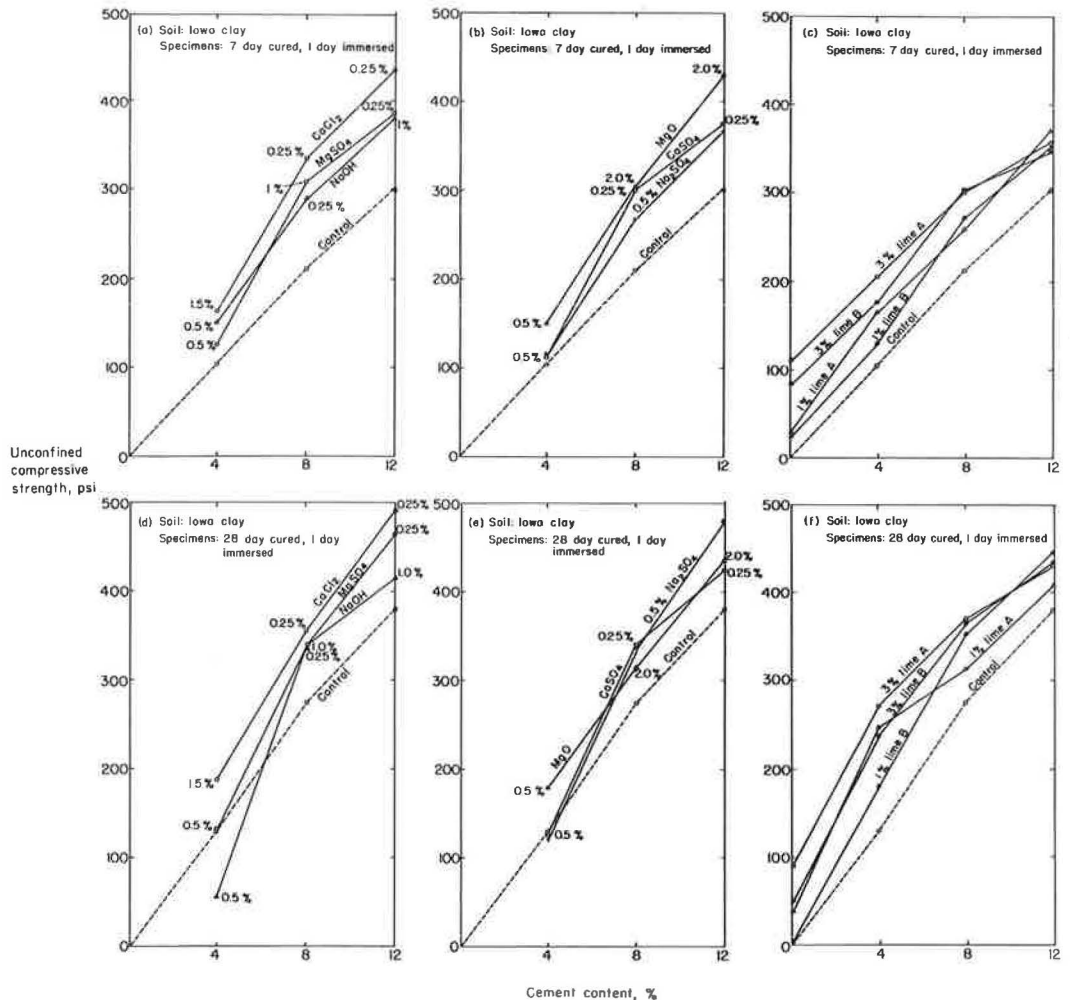


Figure 2. Unconfined compressive strengths of Iowa montmorillonitic, A horizon clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

sulfate, the small ionic radius (0.78 \AA) of magnesium as compared with the large ionic radii of sodium (0.98 \AA) and calcium (0.99 \AA).

Illinois Illite Clay

Although as low as 4 percent cement imparted a strength slightly higher than the minimum acceptable, 250 psi, the possibility of improving the mixture of Illinois clay-cement was investigated by incorporating some chemicals or lime (Fig. 5).

Sodium hydroxide and lime are the only additives which upgrade the quality of the Illinois clay-cement mixtures; the other chemicals investigated seem to have a destructive effect. Treatments with the silicate, carbonate and sulfate of sodium, and with the sulfate and chloride of calcium did not benefit the Illinois clay-cement mixtures, bearing out the thesis that the benefit derived from sodium hydroxide and lime is not due primarily to the addition of sodium and calcium ions. Rather, it seems to be because these ions are added in the form of hydroxide, which attacks the cement and produces an abundance of gel that eventually binds the soil particles. At the same time an interaction takes place between the silica surfaces of the quartz in the soil and the hydroxides, similar to the reaction between a weak acid and a strong base, assuming that the

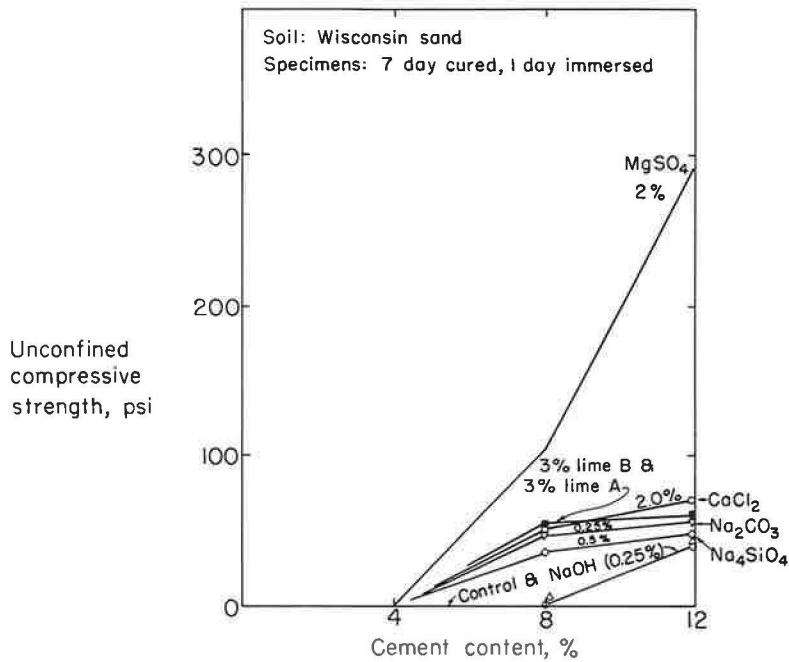


Figure 3. Unconfined compressive strengths of Wisconsin A-horizon pozzolanic sand stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

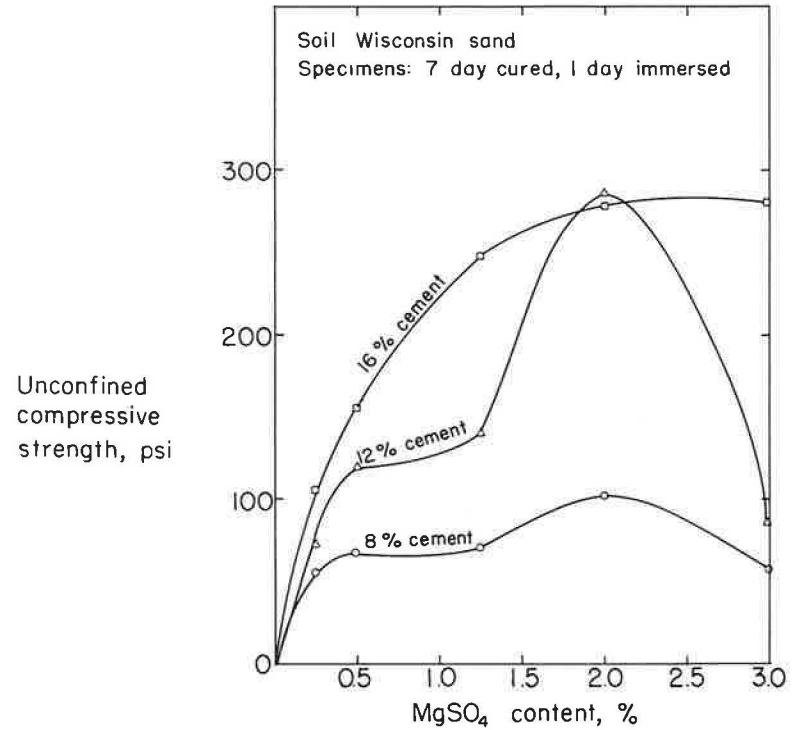


Figure 4. Effect of MgSO₄ concentration on the strength of Wisconsin sand stabilized with cement.

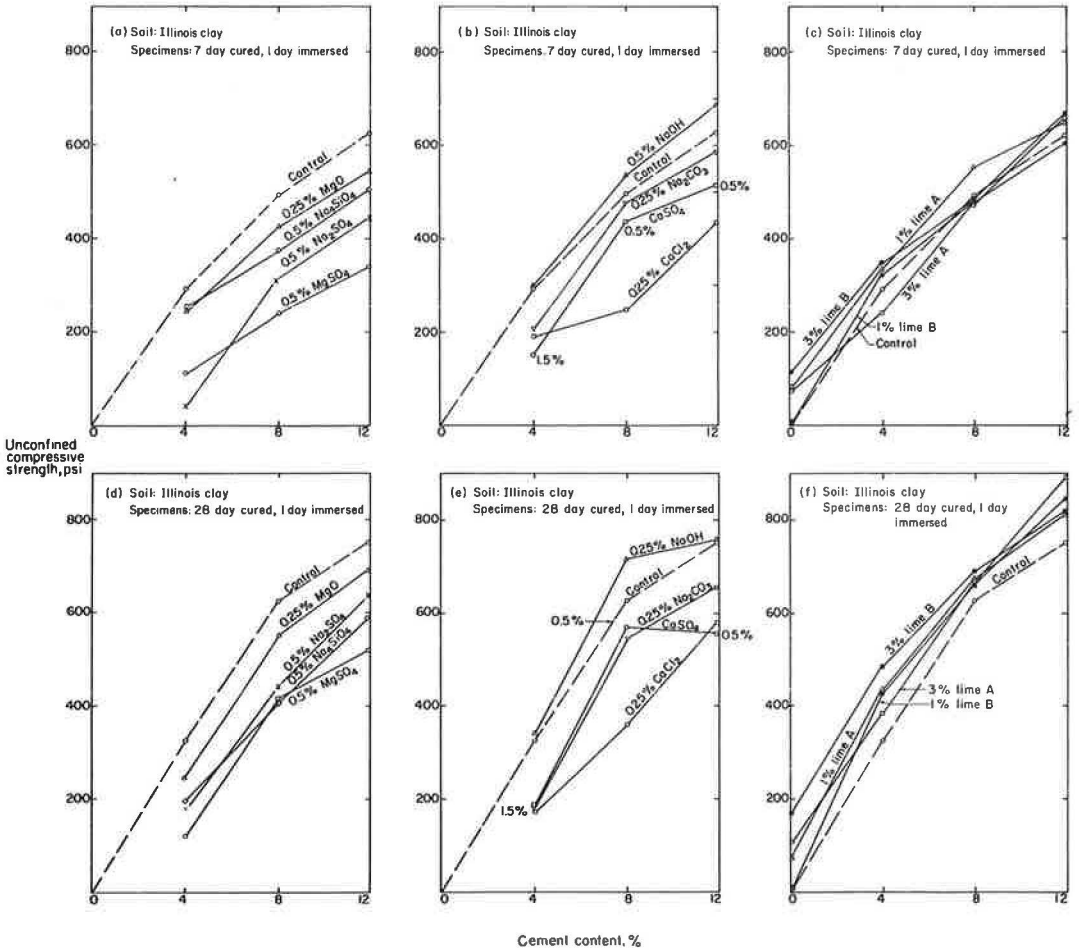
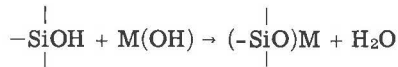


Figure 5. Unconfined compressive strengths of Illinois illitic clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

silica surfaces and quartz are at least partly covered with acidic SiOH groups (9). This reaction may be presented by



in which M stands either for the sodium or calcium ion. Thus, the effects of these two chemicals are centered both in the clay fraction and the quartz surfaces.

Considering that the exchange capacity of the Illinois clay is only 10.8 meq/100g, it would be expected that addition of small amounts of the two effective chemicals would suffice for imparting high strength to Illinois clay-cement mixtures. The optimum amount for sodium hydroxide is 0.25 percent and for lime A (high calcium lime), 1 percent. When 3 percent lime B (dolomitic lime) was used, slightly higher strengths were obtained than with 1 percent. Excess of lime would be deposited as a separate crystalline solid phase (15). Mixtures containing 1.5 percent sodium hydroxide were weaker than those with 0.25 percent and weaker than the control Illinois clay-cement mixtures. It seems possible that the presence of excess cations on the mineral surface causes swelling in the presence of water as these cations tend to dissociate.

Texas Montmorillonite Clay

The response of mixtures of Texas clay-cement to chemical treatments differs slightly from Illinois clay-cement primarily because Texas clay is a heavy clay. The amount of clay-size material is higher, and the predominant clay mineral is montmorillonite; therefore, its exchange capacity is higher. Sodium hydroxide was beneficial at 12 percent cement and at 8 percent cement, the latter only after 28 days curing (Fig. 6) indicating a slower rate of reaction. Although calcitic lime A increased the strength of Texas clay-cement mixtures when used in amounts of 1 percent, it gave higher strengths at 3 percent. Again, the optimum amount of dolomitic lime B was 3 percent.

Of the other sodium compounds used, the orthosilicate improved the strengths when incorporated into mixes containing 8 and 12 percent cement; the carbonate proved effective only in those mixes containing 12 percent cement. The possibility cannot be excluded that these two sodium compounds were effective with Texas but not with Illinois clay because the fineness of the Texas soil provided more available surface area and hence a more extensive chemical reaction. The same reason may also be used to explain the slight benefit derived from mixing calcium chloride with the Texas clay-cement.

Michigan Muscovite-Illite Clay

Figure 7 shows that the mixtures of Michigan clay-cement have an unusual affinity for the chemicals studied. Although the strength improvement from some chemicals

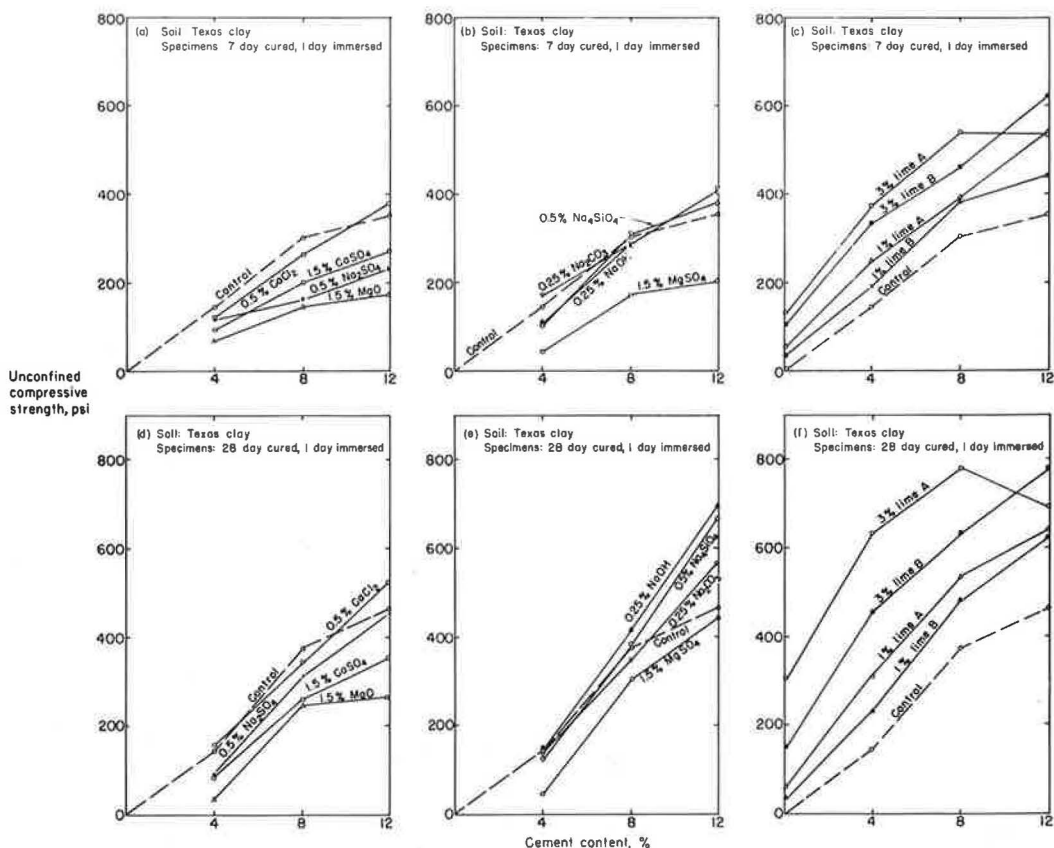


Figure 6. Unconfined compressive strengths of Texas montmorillonitic clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

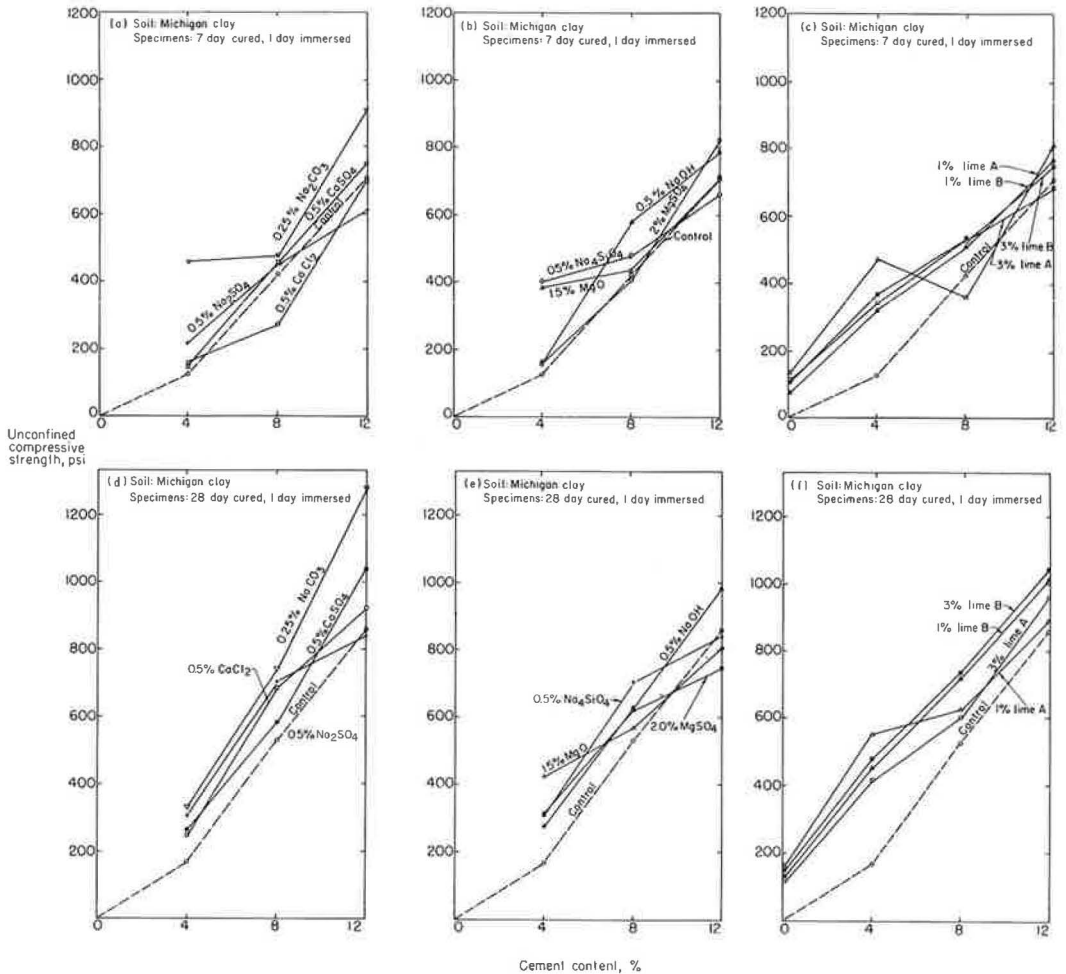


Figure 7. Unconfined compressive strengths of Michigan muscovite-illite clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

may be called erratic, as were the benefits from calcium chloride because of the lack of a definite trend in strength increase, for other chemicals (sodium hydroxide and carbonite) and lime the improvement of the Michigan clay-cement mixtures is most notable. The validity of the suggestion that a low exchange capacity of the soil requires a low concentration of chemical additive to obtain optimum strength conditions is proved by the results with Michigan clay. For sodium hydroxide, calcium sulfate, and sodium carbonate, the optimum amounts were less than 0.5 percent and for lime 1 percent.

Addition of sodium carbonate to Michigan clay-cement mixtures offers a good possibility of producing extreme stability. This stability results from interaction between the calcium of cement and the sodium carbonate, and the subsequent formation of a skeletal calcium carbonate in the soil mass. Also, the release of sodium hydroxide acts in a way to accelerate the soil-cement reaction. There is fairly strong evidence (11) for this explanation. A point of interest emerges from comparing the sodium carbonate treatment of Michigan clay-cement with other clay-cement mixtures, as the latter show a marked decrease in strength. The explanation involves the assumption that the difference in response to sodium carbonate lies in the acidity of the soils. Michigan clay is a nearly neutral soil ($\text{pH} = 6.9$); the other soils are either acid ($\text{pH} < 6$) or alkaline ($\text{pH} > 8$).

North Carolina Kaolinite-Halloysite Clay

The type clay mineral (kaolinite-halloysite intermediate) in the North Carolina clay would seem to indicate that the addition of cement would impart high strength. But the high clay content (67 percent) and the high exchange capacity restrict the development of high strength so that a barely acceptable strength of 284 psi is attained with 12 percent cement (Fig. 8). On the basis of the simplified explanations given for other clay soils, a secondary function of the optimum 3 percent lime admixture seems to produce an electrical neutrality on the mineral surfaces of the soil particles—a neutrality not attained by the insufficient amount of cement originally added.

The similarity of strength results, between the North Carolina clay-cement containing 1.5 percent sodium hydroxide, and that with 3 percent lime, helps explain the anomalous behavior of various sodium compounds and calcium compounds under acid conditions such as in the North Carolina clay (pH = 5.4). As long as the environment is acidic, the precipitation of calcium aluminates and silicates from the cement cannot take place fully because the cement-water solution does not reach saturation. However, the addition of either the sodium or calcium ions in the form of hydroxide produces near neutral solutions that promote the formation of the cementing gel. A somewhat less refined explanation and more remote possibility is the adsorption of hydroxyl ions from the alkaline medium (supplied by the sodium hydroxide and calcium hydroxide of the lime) onto the oxygens of the silica layer by dipolar bonds.

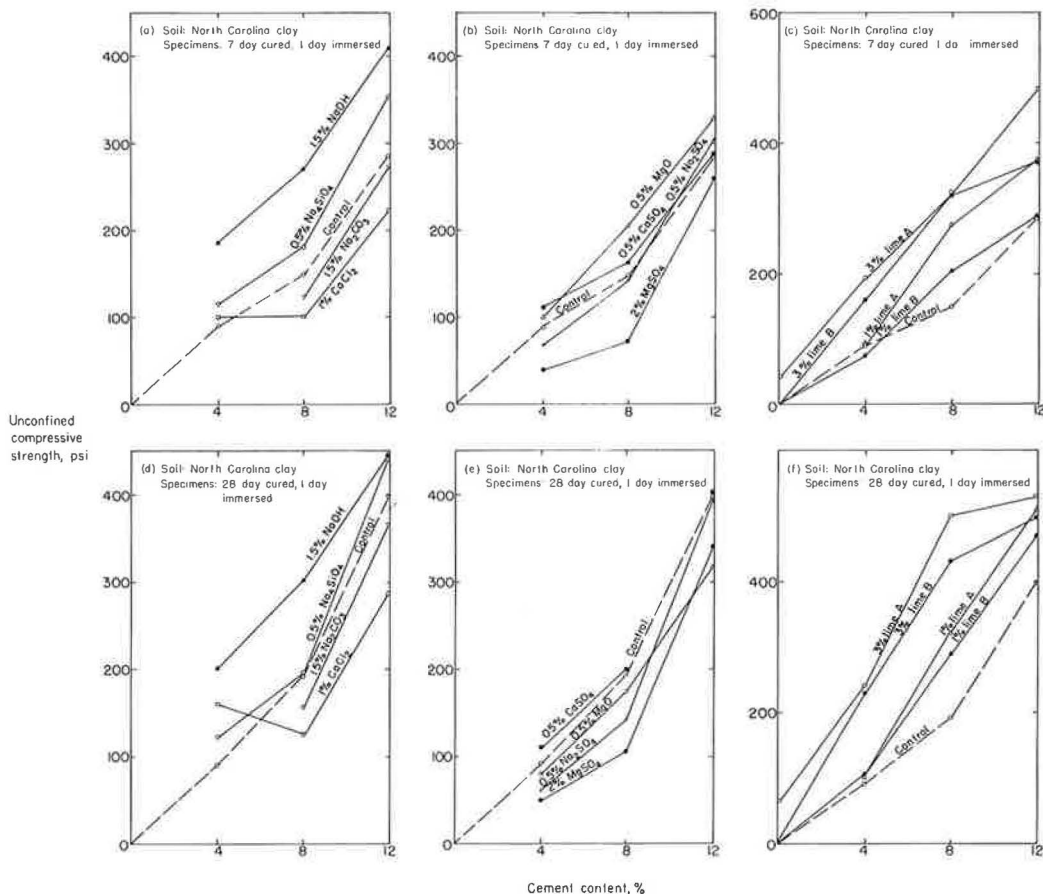


Figure 8. Unconfined compressive strengths of North Carolina kaolinite-halloysite clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

Washington Sand

The high 7-day strengths and the marked difference in 28-day strengths (Fig. 9) between mixtures of Washington sand containing 4, 8, and 12 percent cement are indicative of the concrete-like nature of the mixtures.

That the optimum amount of lime was 3 percent instead of 1 percent shows two separate reactions, cement and sand. This, of course, does not exclude the possibility of a cement-lime synergism, typically occurring in concrete. The assumption that the product resulting from mixing Washington sand with cement is similar to concrete is further substantiated by examining qualitatively the effect of chlorides, hydroxides, and sulfates.

As is generally recognized, alkali chlorides, hydroxides, and sulfates are deleterious to concrete. In areas where, for economic reasons, concrete is made of deleteriously alkali-reactive aggregates the use of a pozzolan has become established practice.

Optimum conditions with the three types of compounds were reached at 0.5 percent for all cement contents. When 1.5 percent of the chemicals was used, the strength decreased. To account for increase in strength when supposedly deleterious chemicals are used the following is offered as an explanation. The extremely small amount of clay (2.5 percent) might have taken up the role of the pozzolan to offset the effect of the sodium and calcium compounds.

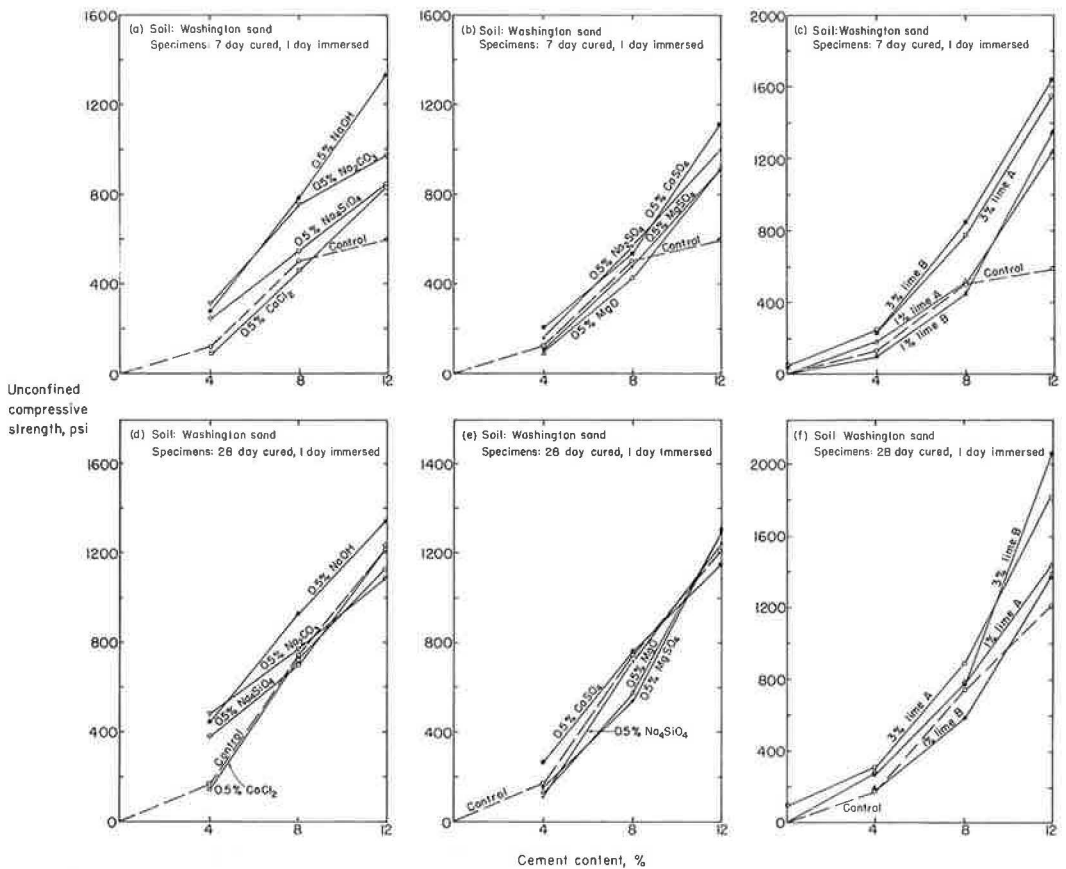


Figure 9. Unconfined compressive strengths of Washington sand stabilized with optimum amount of chemical or hydrated lime at varying cement contents.

DURABILITY TESTS

One of the most important observations regarding the performance of highways is the loss of strength of the base course and especially of the subgrade during spring thaws. There seems no doubt that strength loss originates because of the excessive amount of water absorbed by the soil mass either during a very wet season or during the freezing period. In the latter case the water changes to ice and then reverts to water during the thawing period. Therefore, it may be said that at its in-place moisture content the strength of the soil mass is reduced because its elastic character is partly lost or the soil reaches or tends to reach its plastic state.

The durability indices, R_f and R_i , and the unconfined compressive strengths (Appendix) show how the stabilized soils vary due to weather conditions. For the sake of simplicity in terminology R_f will be referred to as freezing index; it denotes the index of resistance to the effect of freezing, and it is not related to the cumulative degree-day plot (16, p. 126).

Freeze-Thaw Tests

Figures 10 and 11 plot p_{c_i} against p_f values for the stabilized soils. Although a quantitative evaluation of the resistance offered by the stabilized soil to the destructive influence of water or ice is given, the relationships established or deduced can be interpreted only as a general tendency of the particular soil to behave as it did. No attempt has been made to treat the data statistically because the experiment was not planned as such and the data are meager for such a treatment.

From the plots of p_{c_i} versus p_f an algebraic relation may be established

$$p_{c_i} = \frac{p_{c_i} - b}{m} + 250 \quad (4)$$

in which

- p_f = unconfined compressive strength, psi, of specimen cured 7 days and subjected to 10 cycles of freeze-thaw;
- p_{c_i} = unconfined compressive strength, psi, of the 14-day cured specimen;
- b = the p_{c_i} -intercept at $p_f = 250$, psi; and
- m = the slope of the p_{c_i} - p_f line.

This, of course, leads to the establishment of three different functional relationships, a different equation for each cement content (4, 8 and 12 percent). On the other hand, there is the establishment of an overall equation, irrespective of the variations in cement content. The equation may be called the "durability conversion equation." Its practical importance lies in obtaining from the 14-day strength the expected strength loss from freeze-thaw. Thus the same information is obtained as from the freeze-thaw test plus a simplification in equipment and technique. To avoid confusion between the p_f and p_{c_i} as used in the durability tests and in the 14-day strength determinations, the equation may be written

$$q_f = \frac{q_{14} - b}{m} + 250 \quad (5)$$

in which

- q_f = expected strength after 10 cycles of freeze-thaw, psi; and
- q_{14} = unconfined compressive strength, psi, of 14-day cured stabilized soil specimen.

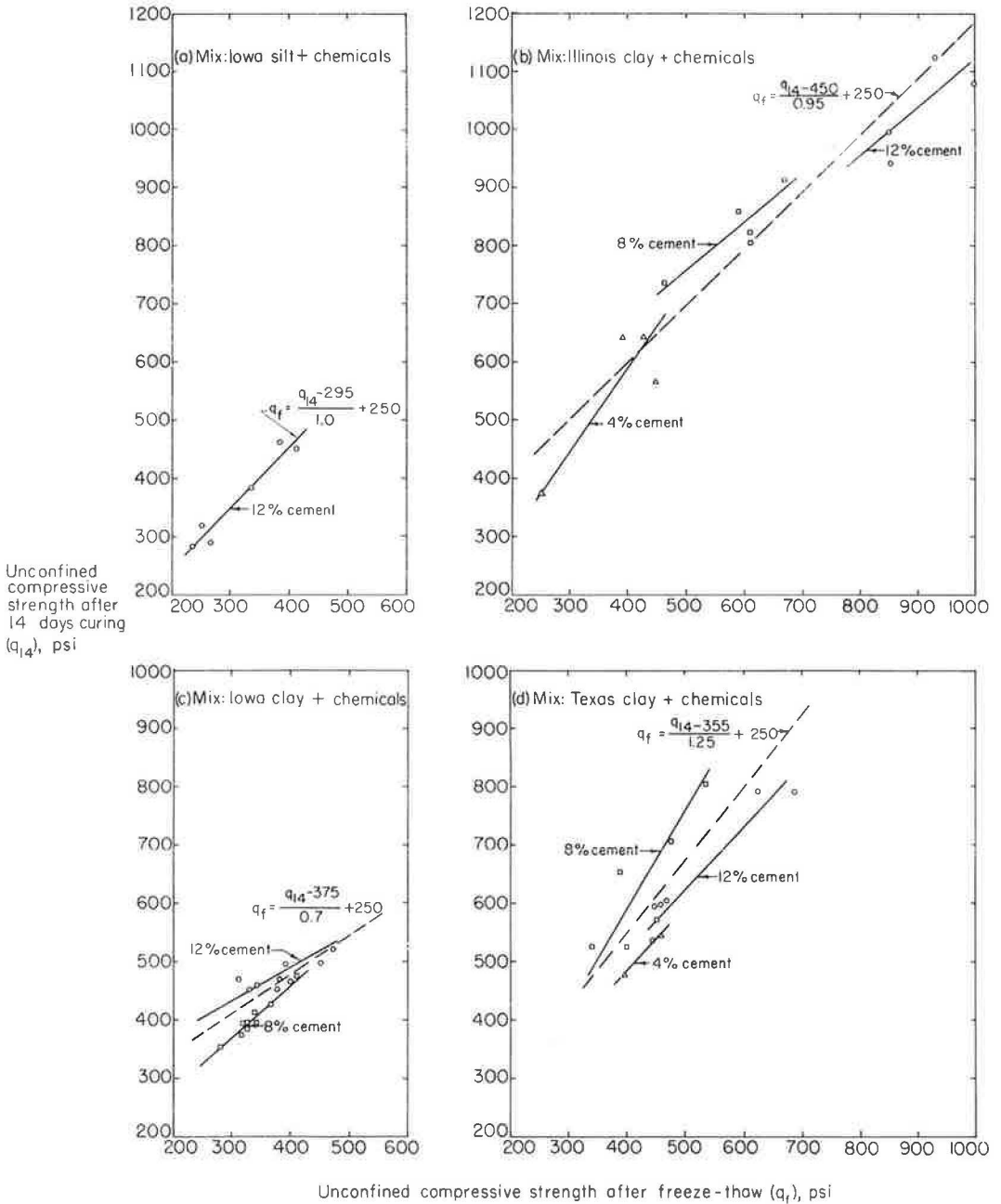


Figure 10. Strength-durability variations in soils stabilized with optimum amount of chemical at varying cement contents.

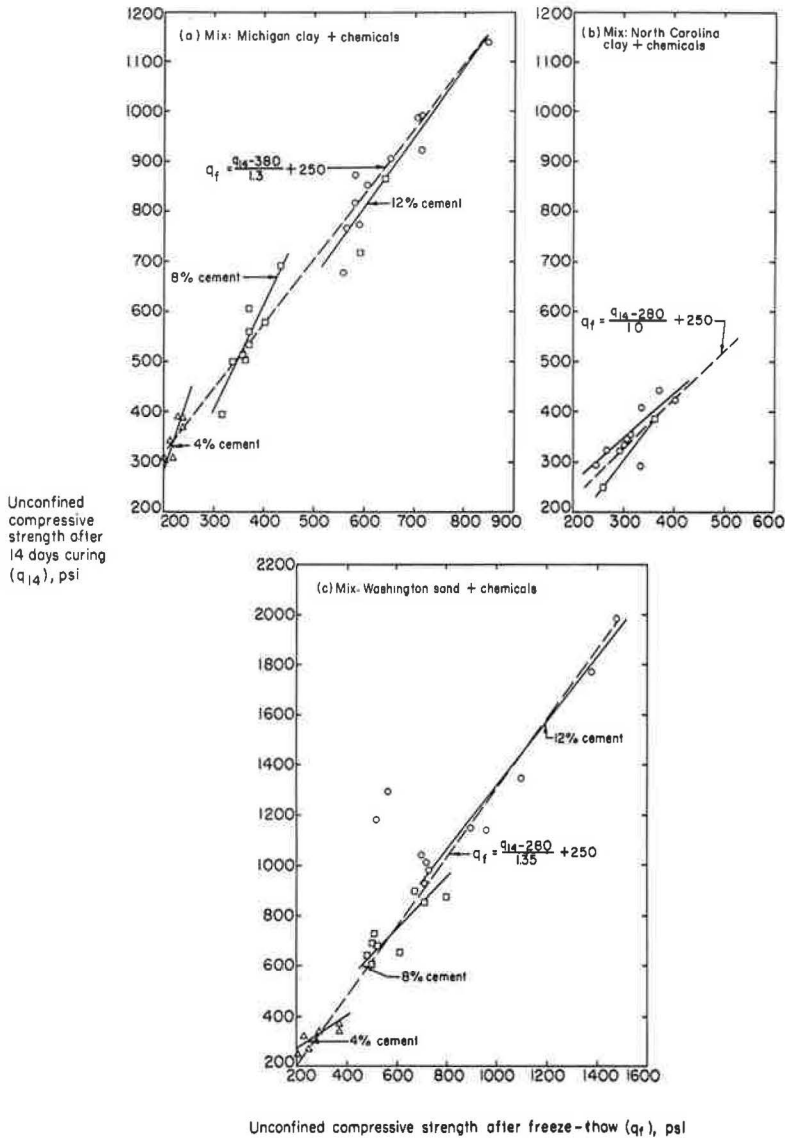


Figure 11. Strength-durability variations in soils stabilized with optimum amount of chemical at varying cement contents.

The equations in Table 4 may give rise to criticisms. Perhaps the most common will be that q_f is a strength value measured 18 days after molding the specimens, while q_{14} represents the strength 14 days after molding. In providing a satisfactory answer, it should be emphasized that the reason for establishing a relationship between freeze-thaw strength values and n -day strength values is to eliminate, if possible, the freeze-thaw test and to develop a means for predicting freeze-thaw reduction in strength.

Another point which needs clarification concerns durability indices less than the minimum acceptable value of 80 percent. When any of the p_f , p_{c1} values are less than the minimum 250 psi, the soil is not adequately stabilized and that particular type of mix should not be used or should be improved. When the values surpass the minimum limit of 250 psi but the durability index is less than 80 percent, then the design value

TABLE 4
DURABILITY CONVERSION EQUATIONS

Stabilized soil type	Cement content, %	$q_f = \frac{q_{14} - b}{m} + 250$	Overall equation
Iowa silt	12	$q_f = \frac{q_{14} - 295}{1.0} + 250$	----
Iowa clay	8	$q_f = \frac{q_{14} - 325}{0.9} + 250$	
	12	$q_f = \frac{q_{14} - 405}{0.55} + 250$	$q_f = \frac{q_{14} - 375}{0.7} + 250$
Illinois clay	4	$q_f = \frac{q_{14} - 375}{1.45} + 250$	
	8	$q_f = \frac{q_{14} - 545}{0.8} + 250$	
	12	$q_f = \frac{q_{14} - 490}{0.8} + 250$	$q_f = \frac{q_{14} - 450}{0.95} + 250$
Texas clay	4	$q_f = \frac{q_{14} - 315}{1.0} + 250$	
	8	$q_f = \frac{q_{14} - 330}{1.7} + 250$	
	12	$q_f = \frac{q_{14} - 350}{1.0} + 250$	$q_f = \frac{q_{14} - 355}{1.25} + 250$
Michigan clay	4	$q_f = \frac{q_{14} - 430}{3.0} + 250$	
	8	$q_f = \frac{q_{14} - 295}{2.15} + 250$	
	12	$q_f = \frac{q_{14} - 320}{1.4} + 250$	$q_f = \frac{q_{14} - 380}{1.3} + 250$
North Carolina clay	8	$q_f = \frac{q_{14} - 240}{1.3} + 250$	
	12	$q_f = \frac{q_{14} - 305}{0.9} + 250$	$q_f = \frac{q_{14} - 280}{1.0} + 250$
Washington sand	4	$q_f = \frac{q_{14} - 305}{0.6} + 250$	
	8	$q_f = \frac{q_{14} - 390}{1.0} + 250$	
	12	$q_f = \frac{q_{14} - 330}{1.3} + 250$	$q_f = \frac{q_{14} - 280}{1.35} + 250$

of the mix is not the least of the 4 values as determined experimentally; the design value to be used is the one adjusted to give a durability index of 80 percent. The adjustment usually calls for a reduction of the experimentally determined strength value.

Some stabilized soil specimens gained strength during the freeze-thaw cycles. Although the resulting strength, p_f was never greater than p_{c1} , it did give freezing index values R_f greater than 100 percent. The freeze-thaw cycles corresponded to a curing treatment. This is true with nearly all Iowa silt specimens, but an exception with clay and sand specimens, and contradicts previous results (13). Freezing in soils is largely a thermal conductivity controlled process, especially in view of the conditions of the Iowa freeze-thaw test where freezing is only from the top surface. To explain the unusual freezing behavior of the stabilized Iowa silt, the basic Stefan mechanism (16, p. 131) is used.

$$z = \sqrt{\frac{48 kF}{L}} \quad (6)$$

Eq. 6 relates the depth of frost penetration z to the thermal conductivity k , the environmental factor F , and the volumetric heat of latent fusion L . Because F is the same for all stabilized soil specimens, the calculated z -values (Table 5) may be used as a basis of comparison. Iowa silt gives the least value. This means that among the soils tested, Iowa silt, in the stabilized form in which it was used and under the conditions imposed by the Iowa freeze-thaw test, is the least affected by frost action. In fact, the depth of

frost penetration is so small that the specimen, except for the bottom $\frac{1}{4}$ in., is in a humid atmosphere, which may account for the gain in strength as compared with that of the fully immersed specimens which reflected either loss of strength or a slower gain in strength. One would normally anticipate that the Wisconsin sand and Washington sand specimens, having high frost penetration depths, should not give freezing index values greater than 100 percent. Out of the 28 specimens of Washington sand tested, three displayed freezing indices greater than 100 percent: two were just above 100 percent, and one was 115 percent. With Wisconsin sand, one out of three specimens gave a freezing index of 110 percent.

Clays with few exceptions indicate freezing indices less than 100 percent.

Therefore, while the Stefan mechanism (Eq. 6) contributes to the understanding of the behavior of Iowa silt and Washington sand, it fails to cover

the other soils where there may have been some interfering effects, possibly of a chemical nature.

In pavement design, the selection of a mix is not based entirely on strength loss due to freezing. Heaving also is taken into account because its extent is compared with that of the permissible deformation in the pavement. At the same time, the rate of heave reflects the permeability of the soil which may be used as an indication of the rate at which water is being moved to the frost line. Since freeze-thaw tests were not run on the natural soils, it is not possible to compare their frost heave with that of their stabilized form. However, measurements on the 2-in. stabilized-soil specimens indicate that the amount of frost heave was never greater than 0.07 in., which corresponds to a rate of heave of less than 0.2 mm per day—a value considered "negligible" according to the U. S. Corps of Engineers frost classification. The small increase in moisture content of the specimens also supports the observation that the specimens evaluated by the Iowa freeze-thaw test may be considered effectively stabilized.

TABLE 5

DEPTH OF FROST PENETRATION OF STABILIZED SOIL

Stabilized Soil	Frost Penetration (in.)
Iowa silt	1.98
Iowa clay	2.02
Wisconsin sand	2.72
Illinois clay	2.19
Texas clay	2.06
Michigan clay	2.10
North Carolina clay	2.00
Washington sand	3.42

TABLE 6
SUMMARY OF STANDARD FREEZE-THAW TESTS

Specimen	Density before freeze-thaw, pcf	Moisture before freeze-thaw, %	Number of cycles completed ^a	Conditions at completion of cycles indicated			
				Density, pcf	Moisture, %	Volume change, %	Stabilized soil loss, %
Iowa silt + 12% cement + 1% lime B	114.4	21.6	10	117.2	27.9	2.4	9.2
Iowa silt + 12% cement + 2% CaSO ₄	114.8	21.5	12	117.1	27.4	2.3	8.7
Iowa clay + 8% cement + 3% lime A	112.2	25.5	8	114.8	29.9	3.1	7.0
Iowa clay + 12% cement + 1% lime B	112.9	25.9	12	114.9	29.9	2.1	7.3
Wisconsin sand + 12% cement + 2% MgSO ₄	114.2	16.5	12	119.4	24.8	0.9	6.4
Illinois clay + 4% cement	131.7	17.9	8	137.9	24.1	2.8	8.9
Illinois clay + 12% cement + 1% lime A	132.7	18.2	12	137.4	23.6	1.1	7.3
Texas clay + 8% cement + 1% lime A	118.0	23.7	6	123.8	30.8	0.8	7.2
Texas clay + 12% cement + 3% lime A	118.5	23.4	12	122.9	27.6	1.2	8.1
Michigan clay + 8% cement + 2% MgSO ₄	125.9	19.7	9	131.2	26.0	1.9	8.3
Michigan clay + 12% cement + 0.25% Na ₂ CO ₃	126.8	19.1	12	130.3	24.9	1.2	8.0
North Carolina clay + 8% cement + 3% lime B	110.7	24.7	7	116.1	29.7	1.2	9.9
North Carolina clay + 12% cement + 3% lime A	110.0	24.9	9	115.8	29.1	1.3	8.4
Washington sand + 4% cement + 0.5% Na ₄ SiO ₄	137.2	11.9	9	140.8	16.5	3.2	10.1
Washington sand + 12% cement + 3% lime A	138.1	12.3	12	140.7	16.6	2.5	9.1

^a See reference (38).

Standard Freeze-Thaw Tests

In comparing the results of the Iowa freeze-thaw test with those of the standard test, (Table 6) it becomes obvious that those mixes which passed the standard ASTM test were considered successfully stabilized on the basis of the Iowa freeze-thaw test, except the North Carolina clay stabilized with 12 percent cement and 3 percent lime A. On the other hand, those mixes which could not withstand the severity of the 12 cycles of the standard freeze-thaw test are not considered as having failed in the Iowa freeze-thaw test.

Although it is unlikely that the results obtained from the two tests can be correlated, the following observation is pertinent. A few unconfined compressive strength tests were run on specimen No. 1 of the standard test at the end of the cycle when failure occurred or after it withstood 12 cycles of freeze-thaw. This specimen was not brushed and was used for volume change calculations. The unconfined compressive strengths determined by the standard test were very low in comparison with the p_f values of the Iowa freeze-thaw test. Perhaps the stabilized soil mixes benefited more, or lost less strength, from the curing nature of the Iowa freeze-thaw test than they did from the standard test. It is surprising that Iowa silt stabilized with 12 percent cement and 2 percent calcium sulfate demonstrated a slightly higher strength in the Iowa freeze-thaw test than after 12 cycles of the standard test. The previously given explanation regarding the depth of frost penetration may account for its unusual behavior.

Because the data collected from the Iowa freeze-thaw test indicate that, in general, those stabilized mixes with an unconfined compressive strength of 250 psi or more at the end of a 7-day humid curing period performed successfully under the imposed conditions, the original hypothesis that the 7-day strength may be used as a dependable guide becomes valid.

Wet-Dry Tests

The results of the wet-dry tests for the stabilized soil specimens (Appendix) may be used as additional assurance of the effectiveness of the stabilization methods used in the study, but not as a design criterion replacing that of the freeze-thaw tests. Such a replacement may be justifiable only in tropical climates which warrant the omission of freeze-thaw testing.

With a few exceptions, the stabilized soil specimens indicated R_i values greater than 80 percent. But even with R_i values less than 80 percent, the strengths of the specimens as reflected by the p_i and p_{c_i} values are greater than the strengths of the corresponding specimens after freeze-thaw testing. Therefore, the wet-dry test results lead to an underdesign, but a design based on the freeze-thaw test results will be safe. Again, the specimens of stabilized Iowa silt showed strengths less than their freeze-thaw counterparts. This should be attributed to the curing rather than the destructive effect of the freeze-thaw test on Iowa silt.

No particular trend was observed in regard to increases of moisture contents of the specimens or was there any tendency of the specimens to absorb too much water during the wet-dry test.

TRIAxIAL COMPRESSION TESTS

In this study, failure is considered to be the loss of shearing resistance at an advanced state of stress to which a soil or a stabilized soil, unsaturated and compacted to near standard Proctor density at optimum moisture content, has been subjected to an exterior load.

Inasmuch as no attempt was made to measure the pore water pressures, the values of the two shear components, cohesion and angle of internal friction, as determined from the Mohr diagrams (Figs. 12 to 16) and given in Table 7 are not the "effective" values.

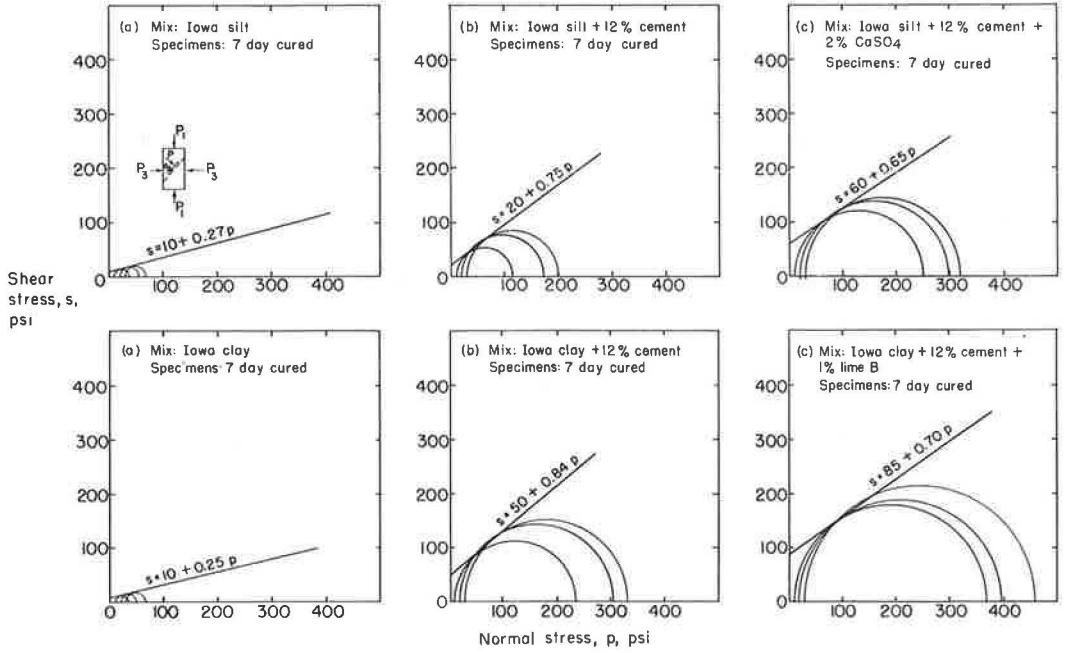


Figure 12. Mohr diagrams for tests on Iowa silt and clay mixes.

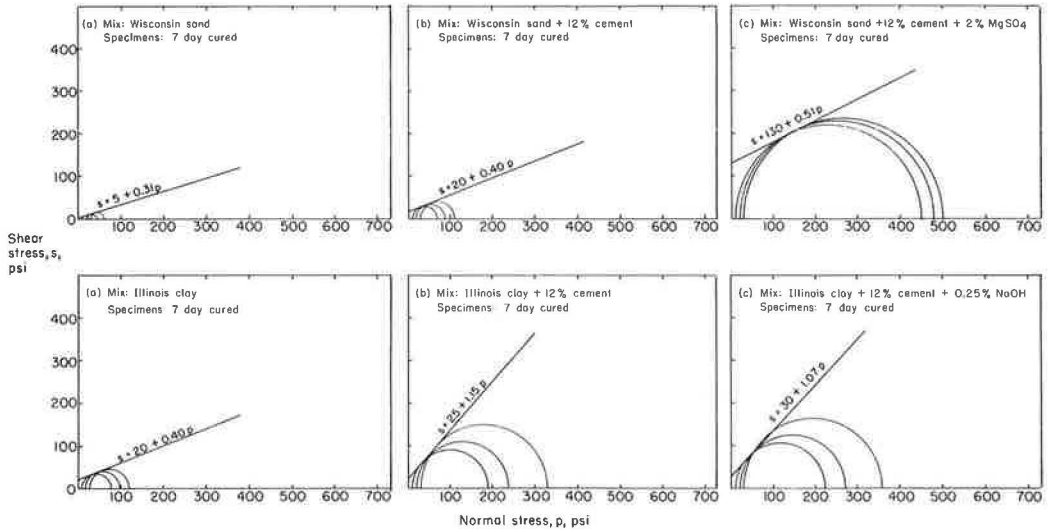


Figure 13. Mohr diagrams for tests on Wisconsin sand and Illinois clay mixes.

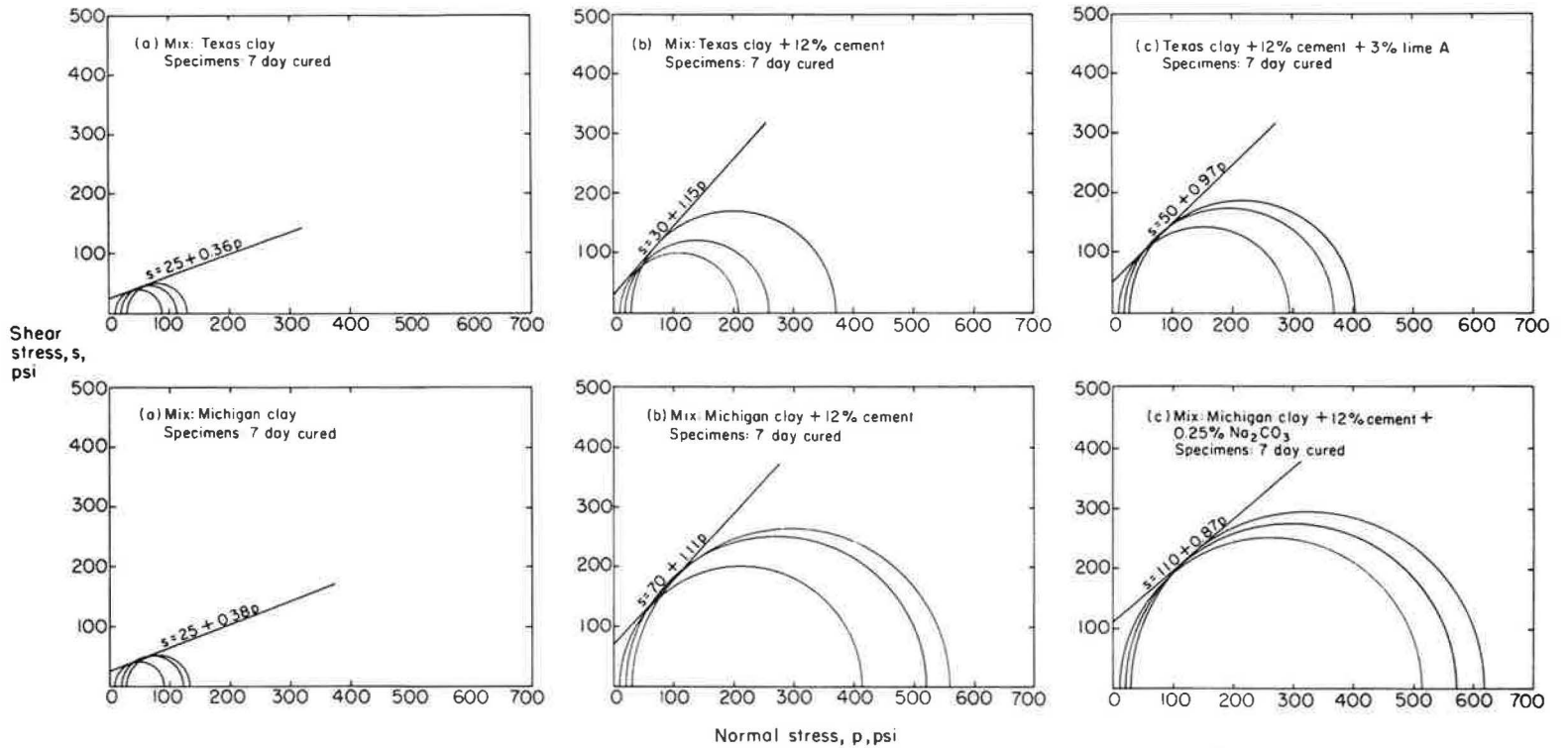


Figure 14. Mohr diagrams for tests on Texas and Michigan clay mixes.

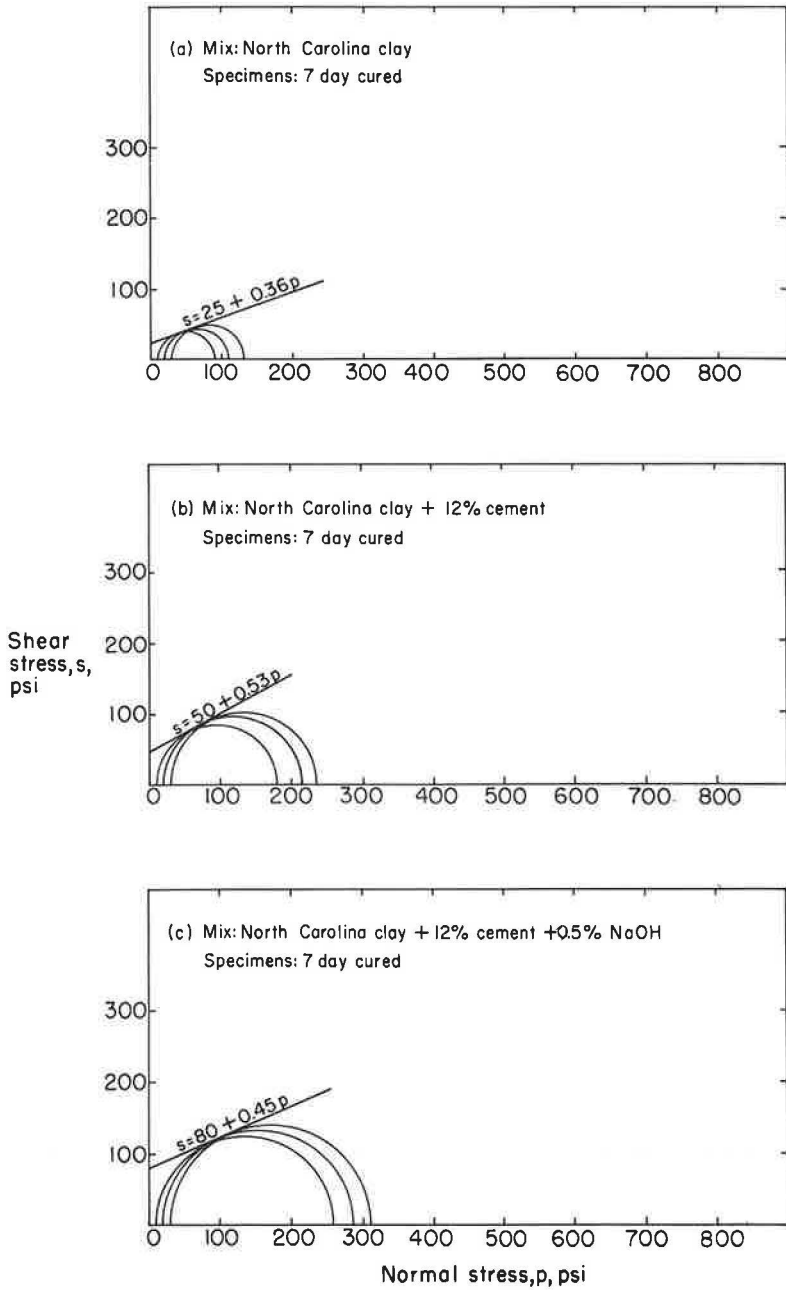


Figure 15. Mohr diagrams for tests on North Carolina clay mixes.

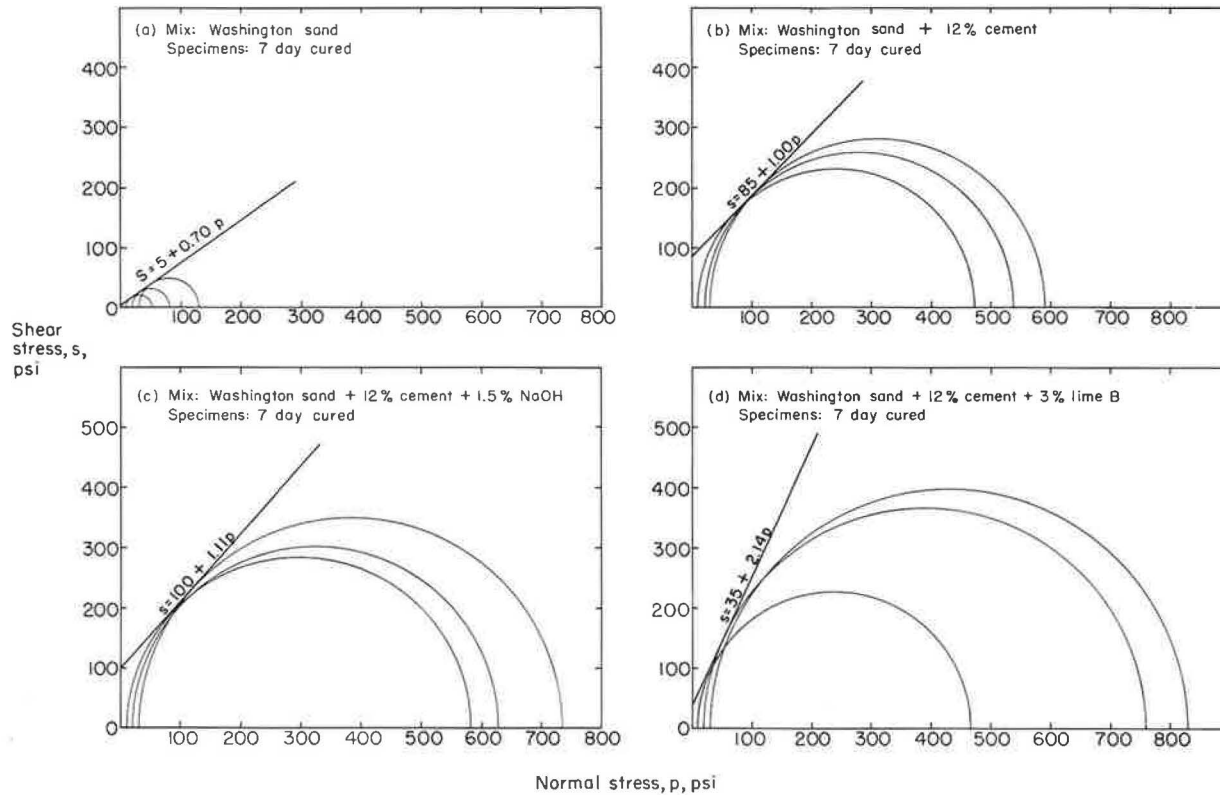


Figure 16. Mohr diagrams for tests on Washington sand mixes.

TABLE 7
SHEAR COMPONENTS OF RAW AND STABILIZED SOILS

Sample designation	Cohesion c, psi	Angle of in- ternal friction ϕ , degrees	Shear equation $s = c + p \tan \phi$
Iowa silt	10	15	$s = 10 + 0.27p$
Iowa silt + 12% cement	20	37	$s = 20 + 0.75p$
Iowa silt + 12% cement + 2% CaSO ₄	60	33	$s = 60 + 0.65p$
Iowa clay	10	14	$s = 10 + 0.25p$
Iowa clay + 12% cement	50	40	$s = 85 + 0.84p$
Iowa clay + 12% cement + 1% lime B	85	35	$s = 85 + 0.70p$
Wisconsin sand	5	17	$s = 5 + 0.31p$
Wisconsin sand + 12% cement	20	22	$s = 20 + 0.40p$
Wisconsin sand + 12% cement + 2% MgSO ₄	130	27	$s = 130 + 0.51p$
Illinois clay	20	22	$s = 20 + 0.40p$
Illinois clay + 12% cement	25	49	$s = 25 + 1.15p$
Illinois clay + 12% cement + 0.25% NaOH	30	47	$s = 30 + 1.07p$
Texas clay	25	20	$s = 25 + 0.36p$
Texas clay + 12% cement	30	49	$s = 30 + 1.15p$
Texas clay + 12% cement + 3% lime B	50	44	$s = 50 + 0.97p$
Michigan clay	25	21	$s = 25 + 0.38p$
Michigan clay + 12% cement	70	48	$s = 70 + 1.11p$
Michigan clay + 12% cement + 0.25% Na ₂ CO ₃	110	41	$s = 110 + 0.87p$
North Carolina clay	25	20	$s = 20 + 0.36p$
North Carolina clay + 12% cement	50	28	$s = 50 + 0.53p$
North Carolina clay + 12% cement + 1.5% NaOH	80	24	$s = 80 + 0.45p$
Washington sand	5	35	$s = 5 + 0.70p$
Washington sand + 12% cement	85	45	$s = 85 + 1.00p$
Washington sand + 12% cement + 1.5% NaOH	100	48	$s = 100 + 1.11p$
Washington sand + 12% cement + 3% lime B	35	65	$s = 35 + 2.14p$

Sandy Soils

The Wisconsin and the Washington sands have very low cohesive strengths of 5 psi in their unstabilized condition. Also, the Washington sand has a 35° angle of internal friction, as expected, but the Wisconsin sand has an unexpected low angle of 17° . The angular variation found between two like soils may be related to their composition. The clay content in the Wisconsin sand is higher by 4 percent. Coupled with a higher molding moisture, this produced a lubricant matrix, a condition not established in the Washington sand. The addition of cement has the same overall effect on both soils. It increased both the cohesion and angle of internal friction. The addition seemed to improve the coarser Washington sand more than it did the finer Wisconsin sand. Apparently cement fills in a large part of the voids in the Washington sand, producing a dense structure where the individual particles are more firmly embedded in a finer cementitious material which restricts their relative movement.

The addition of 2 percent magnesium sulfate and 0.5 percent sodium hydroxide to the Washington sand stabilized with 12 percent cement proved beneficial on the basis of the unconfined compressive strength. The chemicals further improved the shearing strength by increasing the cohesion and the angle of internal friction. This is the natural result of the hypothesis of cementation of granular soils. The chemicals in combination with cement produced a greater amount of gel which either encases the granular particles or acts as a void filler. This has two interpretations: the extent of cementation between particles is greater, and it tends to provide the soil mass with agglomerated finer particles. The first is conducive to greater cohesion, the second to larger values of internal friction.

Clayey Soils

All other soils in this study have been grouped for convenience into the "clayey" category, although they are not classified as clays texturally.

The addition of cement to clayey soils seemed to have the same effect as it had on sandy soils. The cohesive strength of clayey soils improved simultaneously with an increase in the angle of internal friction. The amount of increase varies from soil to soil, but no definite relationship could be established between the soil type and the change in the values of the two shearing components (Table 7). However, when the four inorganic clayey soils, Illinois clay, Texas clay, Michigan clay, and North Carolina clay are considered, the slightly inclined PI- ϕ and PI-c curves of the untreated soil (Fig. 17) become nearly vertical lines when depicting the same soils stabilized with 12 percent cement. In fact, the lines fall in a band limited by P.I. 8 to 19. A decrease in the plasticity index is a manifestation of aggregation, which leads to a larger internal friction angle. The cementation accruing from the cement gel was the factor that led to a higher cohesive strength.

The first and most important observation of the effect of chemical additive on soil-cement when the soil is nongranular is the tendency of the mix to gain in shearing resistance; yet, as Table 7 and Fig. 18 indicate, the increase in cohesive strength is masked somewhat by a slight decrease in the angle of internal friction when chemicals are added to soil-cement. The decrease in internal friction could not be explained in terms of a possible reduced agglomeration of particles at least not so far as the plastic properties were concerned. The soil-cement mixes did not seem to have plasticity indices very different from those of soil-cement modified with chemicals. The reason, then, should lie with the chemical, and the following is a possible explanation. The addition of the chemical gives a higher concentration of electrolyte in the pore fluid with attendant increase in the interparticle attractive forces. This chemical cementation is manifested as an increase in cohesion. But those ions, which are supplied to the soil-cement mass as a chemical, take their place between already aggregated clay particles resulting in a slight increase in spacing between particles. Accommodating the hydrated ion means that the interparticle contact is numerically reduced. It may be reasonable to accept the validity of the proposed hypothesis because internal friction arises from interparticle proximity.

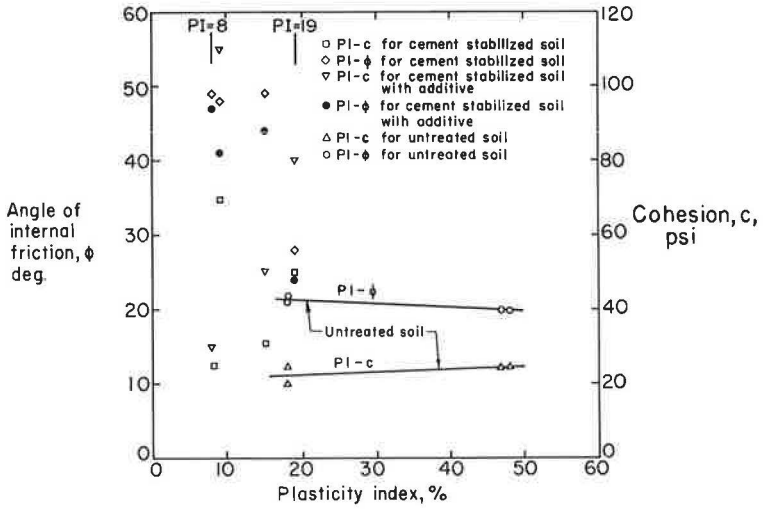


Figure 17. Variation of plasticity index with cohesion and angle of internal friction of untreated and stabilized soils.

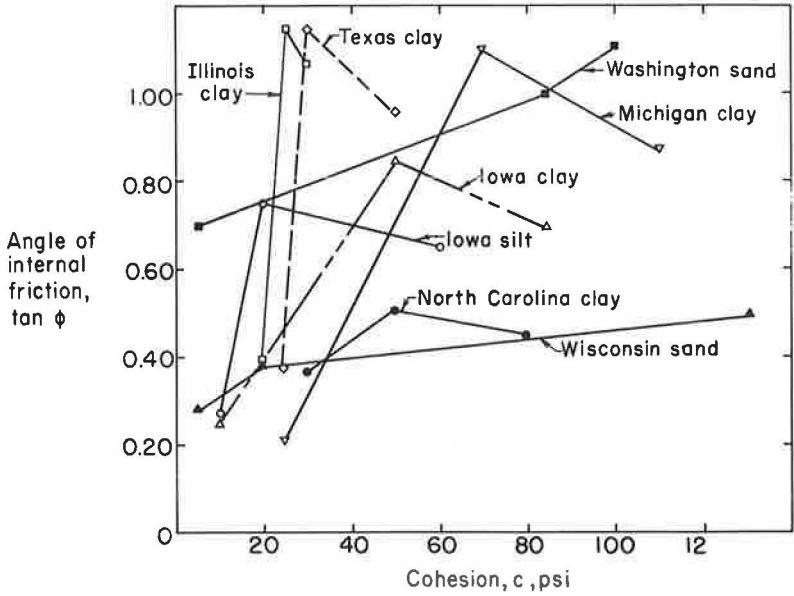


Figure 18. Effect of stabilization on c - $\tan \phi$ relationship of soils.

CONCLUSIONS

Although the following conclusions apply specifically to the 8 soils used in this investigation, the variation in the properties of the soils is such that the conclusions may be considered more widely applicable.

1. The addition of selected chemicals to soil-cement in amounts of not more than 2 percent often results in significant strength increases over that of the soil-cement with equal cement content but no chemical additive. Whether or not the use of chemicals is economical and competitive depends primarily on the local conditions.

2. The type and exact percentage of chemical most beneficial to the strength mixtures of soil-cement are unique for each soil and depend on the texture of the soil, the type of clay mineral, and partly on the acidity of the soil.

3. Organic top soils that are low in clay content and not alkaline (Iowa silt, Wisconsin sand, and Washington sand soil-cement mixtures) responded to treatment with the sulfates of calcium, sodium, or magnesium. Wisconsin sand soil-cement showed a limited favorable reaction to magnesium sulfate only. Sodium hydroxide, and high calcium lime, and monohydrated dolomitic lime were beneficial to soil-cement containing Washington sand but not to Iowa silt or Wisconsin sand soil-cement.

4. The alkaline organic top soil (Iowa clay) in soil-cement did not seem to reflect any special preference for sulfates or hydroxides. The additives increased the strength of the mixtures of Iowa clay-cement but not as much as calcium chloride.

5. Soil-cement mixtures containing the B- and C-horizon soils from Illinois, Texas, Michigan, and North Carolina that are heavy clays texturally, were significantly up-graded strengthwise by additives of lime or sodium hydroxide. Sodium orthosilicate seemed to accelerate the soil-cement reaction in mixtures with all soils except Illinois clay. The beneficial effect of sodium carbonate was limited to mixtures with Texas clay and Michigan clay. Lime seemed especially effective with the montmorillonitic and kaolinitic clays.

These conclusions drawn on the basis of 7-day and 28-day unconfined compressive strength data were verified by the Iowa freeze-thaw and wet-dry test strengths. In almost all cases the assumed minimum acceptable 7-day strength of 250 psi indicated dependability in that mixtures having attained this or higher strength met the strength after freeze-thaw and the freezing index requirements of the test. The same can be said for the wet-dry test results. Therefore, the following can safely be proposed:

1. The Iowa freeze-thaw test is as dependable as the standard ASTM-AASHO freeze-thaw test; in fact, it appears to be a desirable alternate method because of its relative simplicity.

2. The establishment of a functional relationship between the 14-day unconfined compressive strength of the mixture of soil-cement-chemical and the strength of the same mixture after 7 days' humid curing and a full 10-cycle subjection to the Iowa freeze-thaw test further validates the contention that stabilizing agent requirements of soils may be eventually determined by means of simple strength tests.

Triaxial shear tests run on samples compacted to near standard Proctor density at optimum moisture content indicated:

1. The addition of cement increased the cohesion and internal friction angle of the Wisconsin and Washington sands. Magnesium sulfate in the former and sodium hydroxide in the latter further increased the two shear strength components.

2. The other six soils, which fall in the silt-clay textural group, had their angle of internal friction greatly increased and their cohesion only slightly increased upon addition of cement. Chemicals or lime (calcium sulfate with Iowa silt, sodium hydroxide with Illinois clay, monohydrated dolomitic lime with Texas clay or with Iowa clay, sodium carbonate with Michigan clay, sodium hydroxide with North Carolina clay) giving optimum conditions substantially increased cohesion of soil-cement mixtures, but the angle of internal friction decreased slightly, the net result being an overall increase of shearing strength.

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Appendix

TABLE 8
DURABILITY INDICES FOR THE IOWA SILT STABILIZED
WITH 12 PERCENT CEMENT

Additive		P_f	P_{cf}	R_f	P_i	P_{ci}	R_i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
—	—	267	228	117	241	290	83
Calcium chloride	0.50	238	208	114	198	284	70
Calcium sulfate	2.00	412	379	109	409	452	91
Magnesium sulfate	2.00	383	304	126	350	465	75
Sodium sulfate	1.00	337	323	104	313	386	81
Lime B	1.00	251	261	96	254	320	79

TABLE 9
DURABILITY INDICES FOR STABILIZED IOWA CLAY

Additive		P_f	P_{cf}	R_f	P_i	P_{ci}	R_i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 8 Percent Cement							
—	—	337	340	99	337	415	81
Calcium chloride	0.25	323	363	89	392	396	99
Calcium sulfate	0.25	323	369	88	366	396	92
Magnesium oxide	2.00	340	353	96	356	396	90
Magnesium sulfate	1.00	277	297	93	304	353	86
Sodium hydroxide	0.25	313	337	93	343	373	92
Sodium sulfate	0.50	327	337	97	337	383	88
Lime A	3.00	406	409	99	419	475	88
Lime B	3.00	363	389	93	438	429	102
(b) 12 Percent Cement							
—	—	341	340	101	450	460	98
Calcium chloride	0.25	390	397	98	470	496	96
Calcium sulfate	0.25	400	410	98	450	465	97
Magnesium oxide	2.00	376	379	99	360	352	102
Magnesium sulfate	0.25	310	320	97	404	470	86
Sodium hydroxide	1.00	380	385	99	470	470	100
Sodium sulfate	0.50	330	350	94	390	452	86
Lime A	1.00	450	468	96	471	498	95
Lime B	1.00	470	490	96	500	520	96

TABLE 10
DURABILITY INDICES FOR STABILIZED WISCONSIN SAND

Additive		P_f	P_{cf}	R_f	P_i	P_{ci}	R_i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 12 Percent Cement							
—	—	64	80	80	73	73	100
Calcium chloride	2.00	83	96	87	87	77	113
Magnesium sulfate	2.00	406	369	110	379	396	96
(b) 16 Percent Cement							
—	—	70	92	96	78	93	84
Calcium chloride	2.00	90	100	90	93	89	105
Magnesium sulfate	2.00	400	408	98	400	400	100

TABLE 11
DURABILITY INDICES FOR STABILIZED ILLINOIS CLAY

Additive		P_f	P_{cf}	R_f	P_i	P_{ci}	R_i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 4 Percent Cement							
—	—	251	238	106	313	376	83
Calcium chloride	0.25	40	590	7	258	373	69
Sodium hydroxide	0.25	389	455	86	425	639	67
Lime A	1.00	425	419	101	386	642	60
Lime B	3.00	448	455	98	458	567	81
(b) 8 Percent Cement							
—	—	586	531	113	540	860	63
Calcium chloride	0.25	461	534	86	508	735	69
Sodium hydroxide	0.25	609	646	94	692	821	84
Lime A	1.00	665	656	101	686	913	75
Lime B	3.00	610	593	103	686	807	85
(c) 12 Percent Cement							
—	—	850	900	95	870	945	92
Calcium chloride	0.25	752	817	92	784	972	81
Sodium hydroxide	0.25	847	870	98	942	997	95
Lime A	1.00	924	1,092	85	918	1,122	82
Lime B	3.00	1,000	1,058	95	860	1,070	80

TABLE 12
DURABILITY INDICES FOR STABILIZED TEXAS CLAY

Additive		P _f	P _{ci}	R _f	P _i	P _{ci}	R _i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 4 Percent Cement							
—	—	113	156	73	139	261	53
Lime A	3.00	399	360	111	363	478	76
Lime B	3.00	471	461	102	415	544	76
(b) 8 Percent Cement							
—	—	402	432	93	458	521	88
Sodium hydroxide	0.25	399	409	95	498	652	77
Sodium orthosilicate	0.50	343	415	83	396	524	76
Lime A	3.00	537	606	89	649	804	81
Lime B	3.00	475	494	96	475	705	67
(c) 12 Percent Cement							
—	—	455	501	92	492	570	86
Calcium chloride	0.50	455	523	86	452	523	86
Sodium carbonate	0.25	471	468	101	506	504	84
Sodium hydroxide	0.25	450	447	102	483	594	82
Sodium orthosilicate	0.50	460	500	92	503	598	84
Lime A	3.00	687	734	93	674	792	65
Lime B	3.00	623	778	80	653	798	82

TABLE 13
DURABILITY INDICES FOR STABILIZED MICHIGAN CLAY

Additive		P _f	P _{ci}	R _f	P _i	P _{ci}	R _i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 4 Percent Cement							
—	—	138	193	72	158	202	78
Calcium chloride	0.50	123	113	109	139	208	67
Magnesium oxide	1.50	240	254	95	316	370	87
Sodium carbonate	0.25	175	156	106	165	281	59
Sodium hydroxide	0.50	241	241	100	228	389	59
Sodium orthosilicate	0.50	228	189	121	201	307	66
Sodium sulfate	0.50	192	182	106	201	307	66
Lime A	1.00	218	225	97	231	340	68
Lime B	1.00	238	287	89	251	389	65
(b) 8 Percent Cement							
—	—	364	410	89	408	503	81
Calcium chloride	0.50	320	377	85	380	393	97
Calcium sulfate	0.50	368	412	89	400	510	79
Magnesium oxide	1.50	373	454	82	475	535	89
Magnesium sulfate	2.00	340	378	90	410	500	82
Sodium carbonate	0.25	438	478	92	517	691	75
Sodium hydroxide	0.50	405	498	81	513	579	89
Sodium orthosilicate	0.50	376	421	90	467	559	84
Sodium sulfate	0.50	373	424	88	448	603	74
Lime A	1.00	593	702	85	610	718	85
Lime B	1.00	644	748	86	752	867	88
(c) 12 Percent Cement							
—	—	584	617	95	700	873	80
Calcium chloride	0.50	592	615	96	622	778	80
Calcium sulfate	0.50	717	748	96	900	993	91
Magnesium oxide	1.50	565	694	82	734	768	96
Magnesium sulfate	2.00	712	795	90	865	992	87
Sodium carbonate	0.25	848	962	88	1,020	1,140	88
Sodium hydroxide	0.50	717	737	97	807	922	88
Sodium orthosilicate	0.50	563	594	95	657	678	97
Sodium sulfate	0.50	581	620	94	700	819	86
Lime A	1.00	608	716	85	715	853	84
Lime B	1.00	657	810	81	822	907	91

TABLE 14
DURABILITY INDICES FOR STABILIZED NORTH CAROLINA CLAY

Additive		P _f	P _{ci}	R _f	P _i	P _{ci}	R _i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 4 Percent Cement							
—	—	0	0	0	61	78	78
Calcium chloride	1.00	55	88	63	78	90	87
Sodium hydroxide	1.50	72	90	80	93	120	78
(b) 8 Percent Cement							
—	—	104	133	78	140	184	76
Calcium chloride	1.00	58	85	68	75	93	81
Sodium hydroxide	1.50	188	175	103	165	192	86
Lime A	3.00	366	353	104	337	383	88
Lime B	3.00	261	277	94	205	251	82
(c) 12 Percent Cement							
—	—	337	320	105	343	294	117
Calcium chloride	1.00	248	294	85	263	291	90
Calcium sulfate	0.50	292	295	99	316	320	99
Magnesium oxide	0.50	272	309	88	303	320	95
Sodium carbonate	1.50	310	317	98	337	348	97
Sodium hydroxide	1.50	317	313	101	313	363	86
Sodium orthosilicate	0.50	340	340	100	398	410	96
Sodium sulfate	0.50	305	310	99	284	338	84
Lime A	3.00	406	422	96	327	412	79
Lime B	3.00	373	399	94	287	442	65

TABLE 15
DURABILITY INDICES FOR STABILIZED WASHINGTON SAND

Additive		P _f	P _{ci}	R _f	P _i	P _{ci}	R _i
Type	Content (%)	(psi)	(psi)	(%)	(psi)	(psi)	(%)
(a) 4 Percent Cement							
—	—	102	133	77	136	158	86
Calcium chloride	0.50	97	100	97	126	161	80
Sodium carbonate	0.50	396	412	90	389	337	115
Sodium hydroxide	0.50	369	461	80	419	369	114
Sodium orthosilicate	0.50	287	294	98	300	304	99
Sodium sulfate	0.50	198	294	67	235	251	94
Lime A	3.00	244	212	115	169	284	64
Lime B	3.00	231	248	93	287	320	90
(b) 8 Percent Cement							
—	—	501	620	81	702	730	96
Calcium chloride	0.50	504	588	86	512	607	84
Calcium sulfate	0.50	497	523	95	640	690	93
Sodium carbonate	0.50	610	666	92	687	652	105
Sodium hydroxide	0.50	800	780	103	792	874	91
Sodium orthosilicate	0.50	480	592	81	534	642	83
Sodium sulfate	0.50	512	592	87	687	687	100
Lime A	3.00	684	773	89	790	901	88
Lime B	3.00	710	768	93	800	852	94
(c) 12 Percent Cement							
—	—	522	784	67	942	1,183	80
Calcium chloride	0.50	697	840	83	886	1,043	85
Calcium sulfate	0.50	963	1,048	92	1,039	1,142	91
Magnesium oxide	0.50	717	850	84	783	920	85
Magnesium sulfate	0.50	573	628	91	1,282	1,296	99
Sodium carbonate	0.50	723	872	83	824	984	84
Sodium hydroxide	0.50	1,109	1,224	91	1,271	1,352	94
Sodium orthosilicate	0.50	719	958	75	1,008	1,008	100
Sodium sulfate	0.50	902	894	101	912	1,153	79
Lime A	3.00	1,382	1,595	87	1,545	1,770	87
Lime B	3.00	1,472	1,667	89	1,709	1,978	86