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Joakim George Laguros
Iowa State University

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EFFECT OF CHEMICALS ON SOIL-CEMENT STABILIZATION

by

Joakim George Laguros

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Approved:

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Signature was redacted for privacy.

Head of Major Department

Signature was redacted for privacy.

Dean of Graduate College

Iowa State University
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INTRODUCTION

The expansion of road nets, especially in areas where local sources of suitable aggregate deposits are nonexistent or being rapidly depleted, has created conditions that make the use of in-place soils with low load carrying capacity imperative. Mixing Portland cement (to be referred to as cement hereafter) with these in-place soils has widely been used, and has resulted in a product which effectively serves as a subbase, base or even surface course.

Despite the successful performance of soil-cement certain limitations do exist. These accrue from certain features of soils that either render the incorporation of cement to soil ineffective or uneconomical to meet design criteria for strength and durability. To avoid such adverse results, attempts have been focused towards finding chemicals which used in small amounts would enhance the effectiveness of cement, thus upgrading the strength of soil-cement and bringing it within economical boundaries. By the same token, possibilities have been explored of using such chemicals to reduce the quantity of cement required to stabilize soils.

The present investigation is concerned with the determination of the effectiveness of some chemicals as catalysts on soil-cement mixtures, and with attempts to explain the phenomenon of strength gain in these mixtures.

In order to accomplish the stated objectives of the investigation, the program of laboratory tests undertaken comprised:

1. The bracketing of the optimum amount of each chemical that would improve the quality of the soil-cement mixture as indicated by its 7 day cured, 1 day immersed unconfined compressive strength.

2. The further evaluation of promising chemically improved mixes by freeze-thaw and wet-dry durability tests.

3. The quantitative measurement of the effect of the best chemical found on the cohesion and internal friction properties of soil-cement.

SOME IMPORTANT CONTRIBUTIONS TO SOIL-CEMENT TECHNOLOGY

Some 50 years of experimentation with soil-cement have resulted in an abundance of experimental data and a multitude of publications. Therefore, it seems more prudent to present as a background some highly selective examples, which are considered milestones, rather than to attempt a comprehensive review of the subject.

Cement

Stated briefly, cement is the product obtained by pulverizing clinker consisting essentially of hydraulic calcium silicates (7). Of the five types recognized, Type I or General-Use Cement is very commonly employed. Its difference from the other types lies mainly in the amounts of the various components and the presence or the absence of others. In all types of cement the major constituents are tricalcium silicate ($3\ CaO \cdot SiO_2$), dicalcium silicate ($2\ CaO \cdot SiO_2$), tricalcium aluminate ($3\ CaO \cdot Al_2O_3$) and tetracalcium aluminoferrite ($4\ CaO \cdot Al_2O_3 \cdot Fe_2O_3$), generally designated in the literature as C_3S, C_2S, C_3A, and C_4AF, respectively. The following is a typical constituent analysis of Type I Portland cement (11):

Constituent	By weight
C_3S	50 percent
C_2S	25 percent
C_3A	12 percent
C_4AF	8 percent
$CaSO_4$	3 percent
CaO	1 percent
MgO	1 percent

The degree of pulverization is indicated by the requirement that at least 90 percent pass the No. 200 U. S. standard sieve.

References 7 and 30, containing historical and modern information on cement and its properties, are recognized as definitive works and excellent source material in this field.

Hardening of Cement

The physicochemical investigations of the mechanism of the hardening of cement upon the addition of water and subsequent mixing, suggest that hydration follows a course indicated schematically in terms of the chemical compounds in Table 1 (30). The processes of setting, hardening and aging are represented schematically in terms of physical systems and conditions in Table 2 (30).

Table 1. Hydration of cement

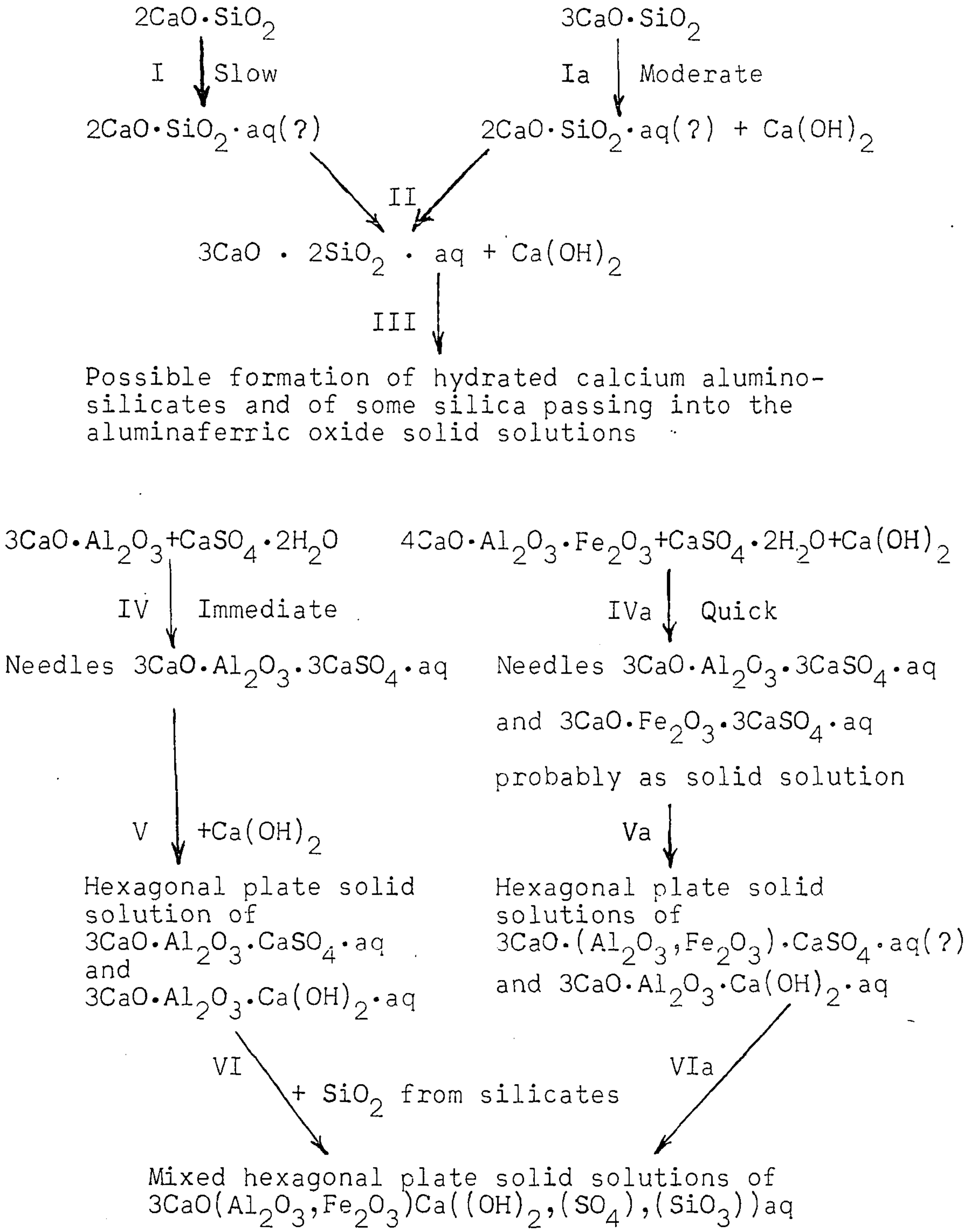
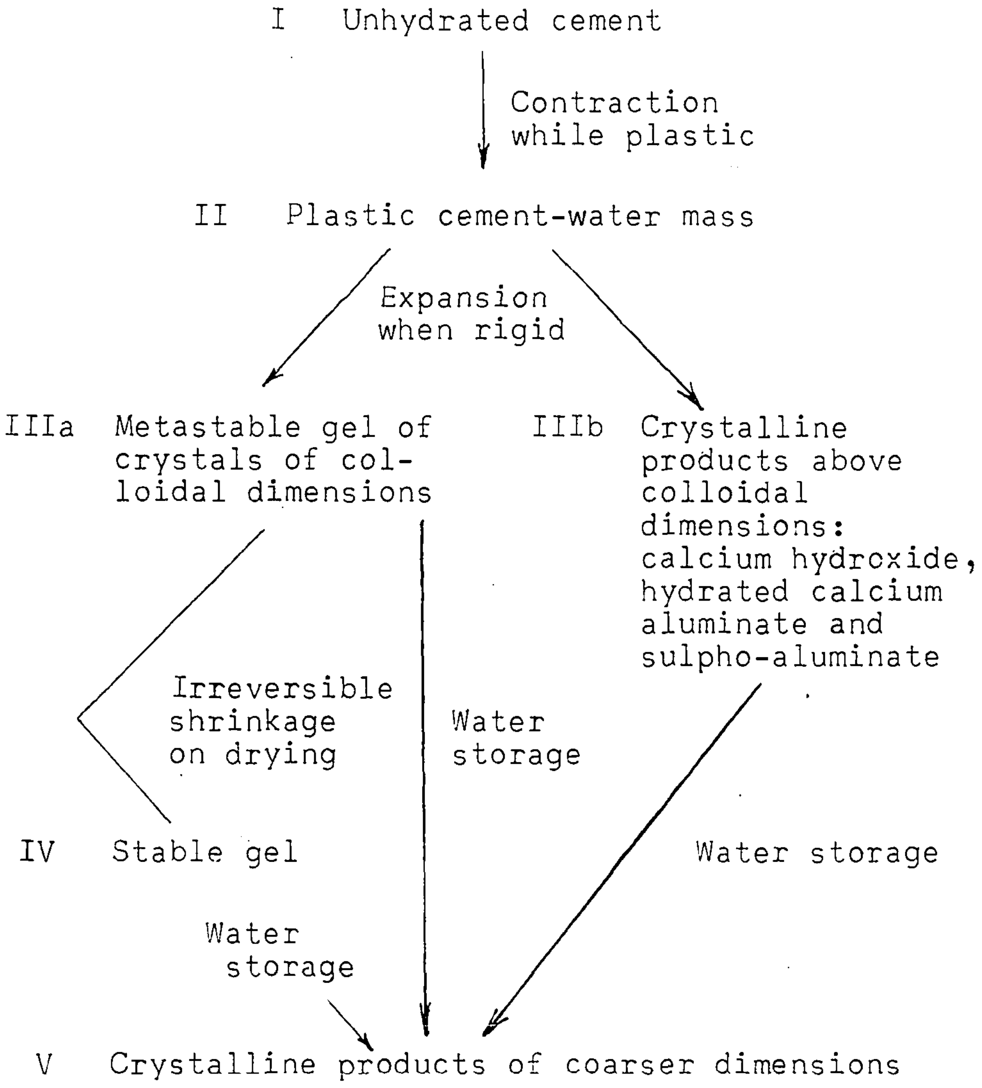


Table 2. Process diagram for cement setting, hardening and aging



Of the reactions indicated in Table 1, the first three (I, Ia, II) have been further investigated by the proton magnetic resonance technique (18). In this work, French and Warder point out that the reaction

$$3CaO \cdot SiO_2 + nH_2O \rightarrow 2CaO \cdot SiO_2(aq) + Ca(OH)_2$$

proceeds rapidly and to a considerable extent in the first few minutes after the addition of water, but it is hindered by the formation of a more or less impervious coating of hydrated dicalcium silicate gel around the reacting particles. Nevertheless, this impervious layer is gradually peptized and the reaction reaches completion. Similarly, it was determined that the reaction involving the dicalcium silicate,

$$2CaO \cdot SiO_2(aq) \rightarrow 3CaO \cdot SiO_2(aq) + Ca(OH)_2$$

occurs simultaneously but at a slower rate. However, both reactions account for the strength attained by the cement pastes, and seem to be in agreement with the evaluation that C_3S is largely responsible for the strength up to 28 days, and C_2S accounts for the long term strength gain (7). At the end of one year, however, the strength contributions of C_3S and C_2S are about equal. Although C_3A also contributes to strength gain, it has a high heat of hydration. To render C_3A inert during early cement hydration, gypsum ($CaSO_4 \cdot 2H_2O$) is added.

The conflicting views as to the nature of the process that eventually gives a hardened paste, seem to have been started by the advocates of the crystallization theory proposed by Le Chatelier on the one hand and by those of the colloid theory advanced by Michaelis on the other hand (cited in Bogue, 7). While Le Chatelier considered the hardening of the set mass to depend on the cohesion and the mutual adhesion of the crystals, Michaelis developed the theory that the hardening of cements is principally due to the formation of colloidal hydration products.

These two theories of cement hydration, divergent as they seemed when first formulated, appear to have many factors in common. The complementary nature of the two theories was brought forth in a compromise suggested by Baykov (cited in Budnikov, 8), whereby cement in the presence of water goes through the processes of solution, colloidal formation, and crystallization.

Hardening of Soil-Cement

When cement is added to soil and the two are thoroughly mixed in the presence of the right amount of water, properties resulting from the change in the structure of the soil conducive to the attainment of strength and durability are introduced into the mixture. One of the first attempts (10) to explain this change states that the soil particles are agglomerated and the agglomerations are linked together to form a new structural material. Recent reports (5, p. 28) on microscopic studies provide the following explanation:

> The cementing material becomes distributed in the soil-cement mass as a latticed soil-cement skeleton with thin films enveloping the microaggregations of the soil. In the presence in the soil of a water-resistant microstructure the specific surface of finely dispersed soils become considerably reduced and consequently, the effectiveness of cement utilization is correspondingly increased. Small admixtures of cement become distributed in the treated soil as separate outcroppings which are not interconnected and do not form a continuous lattice skeleton. In this case the interaction of soil and cement is mainly directed toward increasing cohesion. With increasing cement content a gain in mechanical strength and impermeability due to the formation of a branched soil-cement skeleton and to filling of the pores between the soil aggregations by the individual hydrating particles of cement is realized.

This, in a way, seems to be the acceptance of both the crystallization and colloid theories.

The activity in soil-cement research produced a third theory advanced by Handy (20): that of chemical cementation between the polarized (inert) surfaces of the soil particles and the hydrated cement. Hydroxyl ions from the hydrating cement gel are adsorbed by partially screened silicon ions giving initially weak bonds. With time the surfaces of the soil particles become depolarized (active), and the originally weak bonds get stronger, thus rendering the whole mass stronger.

In reviewing the hypotheses of bonding mechanism between cement paste and aggregate, Munger (36) discusses the surface phenomenon of epitaxy, that is, the development of intergrowth crystals on a common plane when two crystalline materials with similar type of bonding and lattice patterns come in contact.

The use of more refined instrumentation will eventually confirm one or more of the hypotheses of cementation.

Soil Properties in Soil-Cement Mixtures

Textbooks state that soils can be stabilized by adding 7 to 16 percent cement by volume, the low amount being specified for granular soils and the high amount for plastic soils. Often this has erroneously been interpreted to indicate that any soil can be stabilized with cement; it rather means that the majority of soils can be economically stabilized but there are other soils which require more than 16 percent cement to meet design criteria for strength and durability.

As early as 1940, certain physical properties of soils, such as surface area, grain size, and compacted density, were recognized by Catton as affecting the cement requirements (10). Greater insight was gained by Winterkorn's work (44), which related soil hardening by means of cement to certain chemical properties of the external surfaces of soil particles. External surface properties more strongly affect the chemical behavior of materials than any other property. This is to be expected since the surface a solid presents to a liquid, gas or another solid actually consists of a barrier or series of barriers which must be penetrated before chemical reactions can occur. The merit of this work lies in that it recognized the importance of surface phenomena and an attempt was made to correlate them to the engineering behaviour of soils, as displayed by density-moisture relationships, permeability, and freeze-thaw resistance.

Mode of Mixing Soil-Cement

It has been common laboratory practice to mold test specimens immediately after mixing. Field conditions, however, are such that a time lapse occurs between mixing and compaction. When the delay was simulated in the laboratory, lower compressive strengths were obtained for soil-cement and soil-cement-lime mixes, especially with clay soils (17, 43, 13).

Improvement of Soil-Cement Mixes

The structural improvement of soil-cement can be attained by the use of rapid hardening or Type III cement as judged from some recent laboratory results (12).

Aside from using a different type of cement, the effect of some electrolytes on $3CaO \cdot Al_2O_3$, one of the constituents of cement, was studied by Budnikov and Azelitskaya (9). It is of interest to note the dual, in fact the diverse, character of the influence of the electrolytes. On one hand, they retard the hydration of $3CaO \cdot Al_2O_3$, thereby slowing down the formation of "hexagonal platelets" which are unstable and which eventually change into the more stable cubic form. On the other hand, the added electrolytes dissociate the water molecules, which as a result of their dissociation possess higher kinetic energy than the associated molecules, and act more intensively on $3CaO \cdot Al_2O_3$. This accelerates the formation of the "hexagonal platelets". On the evidence from microstructural analysis that the conversion of "hexagonal platelets" to "cubic platelets" is not dependent on the amount of electrolyte added, it was deduced that the greater the number of "hexagonal platelets", the more numerous the "cubic platelets" finally become.

Lime has been used in soil-cement work with the purpose of reducing the plasticity of the soil prior to the addition of cement. This may prolong construction through repetition of construction procedures. The use of lime contents above two percent was considered unjustifiable (35) and it was suggested that the improvement of the quality of the soil mixture is achieved only when the cation of the clay micelle is replaced by a much less hydrophilic added cation.

It has been difficult to cement stabilize certain organic soils because the reduced pH value of these soils causes a precipitation of an alumina-silica gel over the cement particles and inhibits the normal hardening process (34). Because the addition of lime was thought to reduce the acidity of the soil, attempts have been made to use lime, especially when "non-carbonate highly clay soils" were encountered (33). In this study the increase in the compressive strength of the lime soil-cement specimens is explained in terms of the crystal structure, whose chemical composition is said to be similar to that of concrete. However, the effectiveness of lime has been questioned (39) and its deteriorating influence is traced to its low solubility and to the fact that its presence retards the hydration of cement.

The knowledge gained by studying the effect of electrolytes on cement hardening and the benefit that some soils derived from lime treatment led to the use of chemicals as additives to soil-cement mixtures. The limited works by

Handy (20) and Tawes (41) indicate that small amounts of alkali additives in soil-cement accelerate the rate of hardening. The also recent work in the area by Lambe et al. (29) presents similar findings but covers a wide range of chemicals. Of the detailed conclusions drawn, what stands out is the fact that the improvement of soil-cement takes place at low additions of the chemicals - and this will make the practical application economically feasible - and it is permanent.

From the works cited so far it is observed that the attempts were focused at improving soil-cement by incorporating the chemicals in the mix. With the ultimate purpose of increasing the hardness of the upper surface of a compacted roadbase, a laboratory investigation was undertaken (22). The results indicated that sprinkling the surface of soil-cement specimens is effective only when the pozzolanic activity of the loessial soil or mixture is conducive to surface hardening.

Strength-Age Relationship

In determining the cement requirements of soils and in isolating and defining the types of chemicals which could bring about a strength benefit in the soil-cement mixture, it is impossible to ignore the question of whether the early strength can be used as a measure of the long-term strength and durability of the soil-cement mixture. In addition to the 7 day versus 28 day compressive strength correlation (1), which indicates that the average increase in compressive strength at 28 days may be about 30 percent of the 7 day strength, Circeo's (11) extensive statistical study on the subject led to drawing the following conclusions:

1. Soil-cement increases in unconfined compressive strength with time of curing in both a logarithmic and semi-logarithmic manner.

2. The unconfined compressive strength of soil-cement mixtures can be predicted up to curing periods of five years from the standard tests of soil-cement mixtures, using a semi-logarithmic relationship.

3. The slope of the semi-logarithmic strength-age relationship can be used as an indication of the quality of a soil-cement mixture.

Minimum Strength Requirements of Soil-Cement

The amount of cement to effectively stabilize a soil has been determined by durability criteria developed from lengthy experimentation in the Portland Cement Association's laboratories (38). The durability criteria stem from the now standardized wetting-and-drying and freezing-and-thawing tests (2). Because of the destructive nature of these tests, they are viewed as extremely severe and at the same time they deal with surface phenomena to a great extent. The British durability tests (32), however, seem to provide a measure compatible with the changes that take place in the total mass of the soil-cement mixture when it is exposed to adverse weather conditions. In this test, a soil-cement mixture is accepted as satisfactory when a cured specimen molded from it manifests an immersed unconfined compressive strength of 250 psi at the end of seven days. Furthermore, this test requires that the CBR value of the soil-cement mixture be 120 percent. Experience gained in other parts of the world (15) indicates that the above specified values may be, in fact should be, varied according to prevailing climatic conditions. Therefore, this test leaves the determination of its specifics to the judgement and discretion of the designing engineer. Although this test is far better than the one where cement requirements are determined from the permissible weight losses in the freeze-thaw test, it has the inherent weakness of not taking into account the in-situ lateral compression in a soil-cement base or subbase, which tends to increase the shearing resistance of the soil-cement mass. However, until such time as the lateral support to soil-cement can be calculated exactly, the unconfined compressive strength method is the practical alternative.

In devising tests and making them acceptable, the tendency seems to be towards economy of material coupled with simplicity of procedure and shortened time. The use of soil series as a basis for estimating the cement requirements of soils is an evidence of this approach put into practice (31, 21). Also attempts are under way at successful predictions of cement requirements by utilizing surface area measurements of soil particles (14) or by correlating complicated tests to simple ones on a strength basis (16).

The Triaxial Compression Test

For years the stability of soil has been measured in terms of its cohesion and angle of internal friction. The direct measurement of cohesion and internal angle of friction is difficult, but the use of the triaxial compression test, and of its results presented graphically by the Mohr diagram, provides the possibility of rather accurately determining these two properties (6, 4). The shearing resistance or strength, s, of the soil tested in triaxial compression is determined by Coulomb's equation (42)

$$s = c + p \tan \phi$$

where

p = normal stress on the surface of sliding

c = cohesion

ϕ = angle of internal friction

The above equation indicates a straight line although the actual locus of points is an envelope and is given by the equations (42)

$$p = \tfrac{1}{2}(p_1 + p_3) + \tfrac{1}{2}(p_1 - p_3) \cos 2\alpha$$

and

$$s = \tfrac{1}{2}(p_1 - p_3) \sin 2\alpha$$

where

p_1 = the total vertical stress

p_3 = the lateral stress

α = angle of shearing plane at failure with the horizontal

As better understanding was gained of the complexity of the soils, the complexity accruing from the fact that soil consists of solid particles, water, and air, it was formulated (6) that both the cohesion and the angle of internal friction are not constants but they vary with the moisture content and the degree of compaction of the soil. Furthermore, research revealed that the shearing resistance of a soil under stress develops as a result of the grain to grain contact only, and the pore water pressure in the soil contributes negatively to its shearing resistance. Thus, Coulomb's equation appears today, in its modified form, as

$$s = C_e + (p - u) \tan \phi_e$$

where

C_e = effective cohesion

u = porewater pressure

ϕ_e = effective angle of internal friction

Reference 25 presents excellent information on the subject.

The recognition of soil-cement as a construction material made imperative the use of triaxial compression tests with the understanding, as Balmer puts it (3,4), that the test is "an additional tool for evaluating the physical characteristics" of soil-cement and not a substitute for the durability tests. The work cited above tends to indicate that the cohesion as well as the angle of internal friction increases when soils are treated with cement, the increase being higher for the granular soils than for the fine grained soils.

MATERIALS

Soils

The eight soils used in this investigation were selected in such a way 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. Figure 1 gives property information on the eight soils, which were sampled from various parts of the Country, and which had already been classified as difficult to stabilize or "problem" soils.

Cement

Type I Portland cement, whose properties are given in Table 3, was used throughout this study.

Hydrated Lime

Two types of hydrated lime were used. One was hydrated high calcium, lime A, and the other was hydrated dolomitic, lime B. Pertinent information on the two limes is given in Table 4.

Figure 1. Properties of soils

Figure footnotes:

[a] Textural gradation tests were performed only on the soil fraction passing the No. 10 sieve. ASTM Method 422-54T.

[b] ASTM Method D423-54T.

[c] ASTM Method D424-54T.

[d] Cation exchange capacity determined by the ammonium acetate (pH = 7) method on soil fraction less than 0.42 mm.

[e] Versenate method for total 1N HCl soluble calcium.

[f] Glass electrode method using suspension of 15 g soil in 30 cc distilled water.

[g] Potassium dichromate method.

[h] X-ray diffraction analysis.

[i] Q standing for quartz.

[j] F standing for feldspar.

[k] M standing for montmorillonite.

[l] Chl-M standing for chlorite-montmorillonite interlayer.

[m] I standing for illite.

[n] Musc-Ill standing for muscovite-illite interlayer.

[o] K-H standing for kaolinite-halloysite intermediate.

[p] Triangular chart, U. S. Bureau of Public Roads.

[q] AASHO Method M145-491.

Sample designation	Iowa silt	Iowa clay	Wisconsin sand	Illinois clay	Texas clay	Michigan clay	North Carolina clay	Washington sand
Sampling location	Wayne Co., Iowa	Calhoun Co., Iowa	Racine Co., Wisconsin	Livingston Co., Illinois	Harris Co., Texas	Ingham Co., Michigan	Durham Co., North Carolina	Snohomish Co., Washington
Parent material	Leached fine-textured Wisconsin-age loess	Surficial sediment derived from Wisconsin-age drift	Glacio-fluvial deposit	Wisconsin-age glacial till	Coastal Plain deposit largely deltaic	Wisconsin-age glacial till	Triassic sediments	Cemented gravelly till
Great soil group	Planosol	Wiesenboden	Gray-Brown Podzolic	Humic-Gley	Grumusol	Gray-Brown Podzolic	Red-Yellow Podzolic	Brown Podzolic
Soil series	Edina	Webster	Plainfield	Clarence (Rowe)	Lake Charles	Miami	White Store	Alderwood
Horizon	A	A	A	C	C	B	B	-
Sampling depth, in.	0-12	0-12	0-120	46-56	39-144	12-36	18-21	0-120
Textural composition[a]								
Gravel (above 2mm), %	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sand (2-0.074 mm), %	5.5	13.5	77.0	10.0	3.0	14.0	13.0	75.0
Silt (0.074-0.005 mm), %	68.5	39.5	13.0	38.0	36.0	29.0	22.0	19.0
Clay (below 0.005 mm), %	26.0	47.0	10.0	52.0	61.0	57.0	65.0	6.0
Clay (below 0.002 mm), %	12.0	28.0	4.0	34.0	40.0	35.0	50.0	0.0
Physical properties								
Liquid limit[b], %	35	58	NP	36	65	43	74	NP
Plastic limit[c], %	27	31		18	18	25	26	
Plasticity index, %	8	27		18	47	18	48	
Chemical properties								
C.E.C.[d], m.e./100g	19.62	40.00	6.63	10.80	27.30	15.06	36.20	3.32
Carbonates[e], %	0.85	1.50	0.50	22.50	16.60	3.30	0.94	0.20
pH[f]	5.3	7.5	7.0	8.3	8.2	6.9	5.4	6.0
Organic matter[g], %	3.56	7.00	1.90	0.70	0.13	0.85	0.27	1.80
Non-clay minerals[h]	Q[i], F[j]	Q, F	Q, F	Q, F	Q, F	Q, F	Q, F	Q, F
Predominant clay mineral[h]	M[k]	M	Chl-M[l]	I[m]	M	Musc-Ill[n]	K-H[o]	Chl-M
Classification								
Textural[p]	Silty clay loam	Clay	Sandy loam	Clay	Clay	Clay	Clay	Sandy loam
Engineering (AASHO)[q]	A-4(8)	A-7-5(18)	A-3	A-6(11)	A-7-6(20)	A-7-6(12)	A-7-6(20)	A-3

Table 3. Properties of the Type I Portland cement used

<u>Chemical analysis</u>	
Silicon dioxide, SiO_2	21.62%
Aluminum oxide, Al_2O_3	5.05%
Ferric oxide, Fe_2O_3	2.97%
Calcium oxide, CaO	64.05%
Magnesium oxide, MgO	2.90%
Sulfuric trioxide, SO_3	2.26%
Insoluble residue	0.16%
Loss on ignition	0.58%
<u>Physical properties</u>	
Fineness, turbidimeter	1355 sq cm/g
Fineness, air permeability	3395 sq cm/g
Compressive strength 1:2.75 G.O.S.	
3 day	2269 psi
7 day	3721 psi
28 day	5625 psi
Soundness, autoclave expansion	0.120%
Setting time, Gillmore needle	2.0 hours

Table 4. Properties of the hydrated limes used

Type	High calcium	Dolomitic monohydrate, Type N
Laboratory designation	Lime A	Lime B
Trade name	Kemikal hydrated	Kemidol hydrated
Processing location	New Braunfels, Texas	Genoa, Ohio
<u>Chemical analysis</u>[a]		
Silicon dioxide	0.28	0.4
Iron and aluminum oxide	0.6	0.3
Magnesium oxide	0.59	31.8
Sulfur trioxide	0.25	1.1
Carbon dioxide	--	1.0
Total calcium oxide	73.82	48.8
Available calcium oxide	70.3	47.1
Loss on ignition	24.1	17.0
Combined H_2O	--	18.0

[a] All values given are in percent by weight.

Chemicals

The selection of chemical compounds investigated was dictated primarily by the consideration that a similarity existed between these compounds and the product resulting from the combination of cement and soil, and secondly by the research findings of earlier studies. Table 5 gives a list of these compounds.

Table 5. Chemicals used

Chemical	Formula	Source
Sodium hydroxide	$NaOH$	Dow[a]
Calcium sulfate	$CaSO_4$	Reagent grade
Magnesium sulfate	$MgSO_4 \cdot 7H_2O$	Analytical reagent
Sodium sulfate	Na_2SO_4	Reagent grade
Calcium chloride	$CaCl_2$	Dow[a]
Magnesium oxide	MgO (C-1-60)	Dow[a]
Sodium orthosilicate	Na_4SiO_4	Dow[a]
Sodium carbonate	Na_2CO_3	Dow[a]

[a] Supplied by the Dow Chemical Company, Freeport, Texas.

METHODS OF INVESTIGATION

In soil stabilization mix design the preliminary phase of the design consists of determining the amount of stabilizing agent which imparts to the soil either maximum strength or adequate strength to meet design criteria. In either case, the economic feasibility of the mixture is taken into consideration.

Preparation of Mixes

A predetermined amount of pulverized air dry soil, passing the No. 10 U. S. standard sieve, was hand mixed with a predetermined amount of cement or lime depending on whether the former or the latter was the main stabilizing agent. Then, the mixture was placed in the mixing bowl, some of the compaction water, which had been calculated to give near standard Proctor density, was added and hand mixed again, and then mixed with a Hobart Kitchen mixer, Model C-100, at low speed, for one minute. Following this, the rest of the compaction water, in which a predetermined 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 hand mixed and finally machine mixed for an additional minute.

It should be emphasized here that the amount of molding water used was the one to give maximum density and not maximum strength. Recent research (37) had indicated that for cement-treated soils the moisture content to produce maximum density was different from the one to give maximum strength, but that the small difference, ± 2 percent maximum, could be ignored for all practical purposes. Also, it should be noted that the molding water contents used in this investigation were those producing maximum density for soil-cement mixtures without chemical additives despite the fact that chemical compounds may tend to introduce some changes in the optimum moisture content for maximum density. Tables 6 and 7 depict the optimum moisture contents used in this study to give maximum density in soil-cement and soil-lime mixtures, respectively. The following example illustrates calculation of amounts of materials for a soil-cement-chemical mix:

Air dry weight of soil	1200	g
Hygroscopic moisture correction factor	0.02	
Oven dry weight of soil	1176	g
Weight of hygroscopic moisture	24	g
Optimum moisture content	16.5	percent
Moisture added for mixing loss	0.5	percent
Total moisture content	17.0	percent
Weight of cement (12 percent)	141	g
Weight of $MgSO_4$ (1 percent)	11.76	g
Amount of water added $(1176 + 141 + 11.76) \frac{17}{100} - 24$	202	g

Table 6. 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

Table 7. Optimum moisture-maximum dry density values of soil-lime mixes

Soil type	Optimum moisture content, %	Maximum dry density, pcf
Iowa silt	22.0	94.4
Iowa clay	24.6	95.1
Wisconsin sand	17.0	100.0
Illinois clay	18.2	111.7
Texas clay	22.3	102.9
Michigan clay	19.7	105.8
North Carolina clay	25.8	97.7
Washington sand	12.2	121.5

The amounts of stabilizing agents are based on the oven dry weight of the soil and the 0.5 percent of water added for mixing loss, although arbitrary as it may seem, is based on experience.

The normal concentration of magnesium sulfate, based on the total amount of water present in the soil-cement mixture, can be calculated from the equivalent weight of magnesium sulfate, $\frac{120.39}{2}$ g equivalents, and the specific gravity of aqueous magnesium sulfate solution. The amount of magnesium sulfate in the molding water is

$$\frac{11.76}{226} \times 100 = 5.204 \text{ percent}$$

This gives a specific gravity of 54.84 g per liter at $\frac{20°}{4°}$C (23, p. 2018). Therefore, a normality of

$$\frac{54.84}{120.39} \times 2 = 0.91 \text{ N}$$

is obtained. The concentration of magnesium sulfate expressed in moles per 100 g of soil, then, is:

$$\frac{1}{120.39} = 0.0083$$

or 8.3 milliequivalents.

Molding

Immediately after mixing, the bowl was covered with a damp cloth to prevent evaporation. Two inch diameter by two inch high specimens were molded in an apparatus developed at the Iowa Engineering Experiment Station, which is shown in Figure 2. The molding procedure used with this double plunger, drop hammer molding apparatus (13) gives specimens with a density near standard Proctor density.

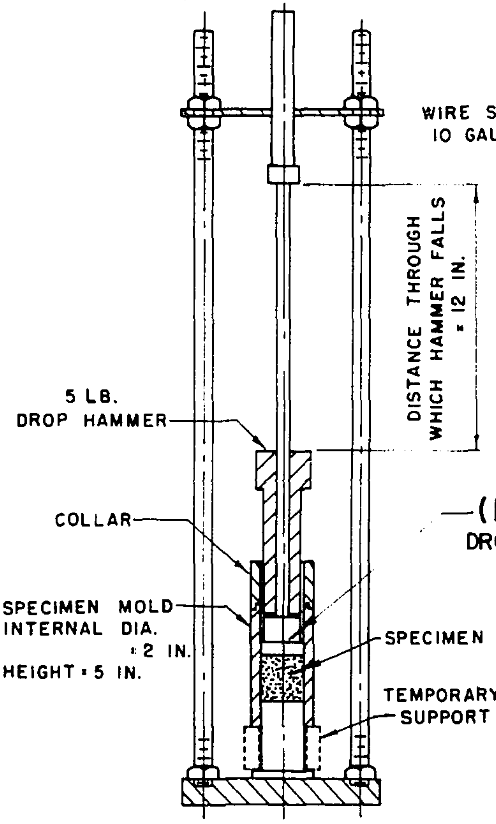
Curing

Immediately after being weighed and measured, the specimens were wrapped in waxed paper and sealed with Scotch tape to prevent loss of moisture. The specimens were then stored in a curing room at a relative humidity of 95 ± 5 percent and a temperature of 70 ± 5°F. Two curing periods were used: 7 days and 28 days.

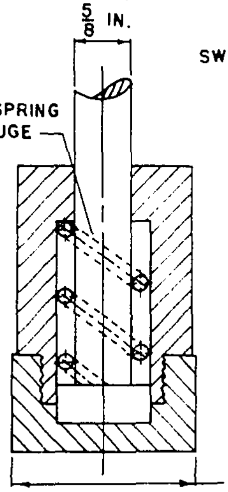
Compressive Strength Testing

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. Specimens were tested, in the testing machine shown in Figure 3, to failure to determine their unconfined compressive strength.

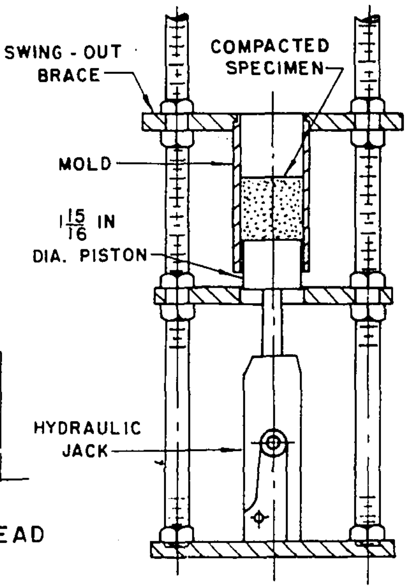
Figure 2. Molding apparatus



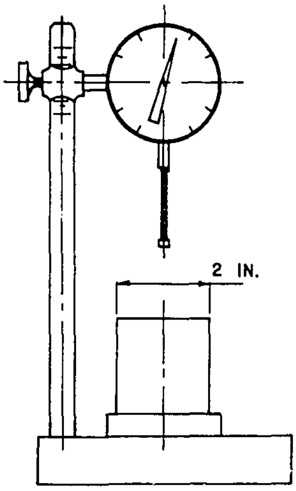
(a) COMPACTION APPARATUS



(b) DETAIL OF DROP HAMMER HEAD



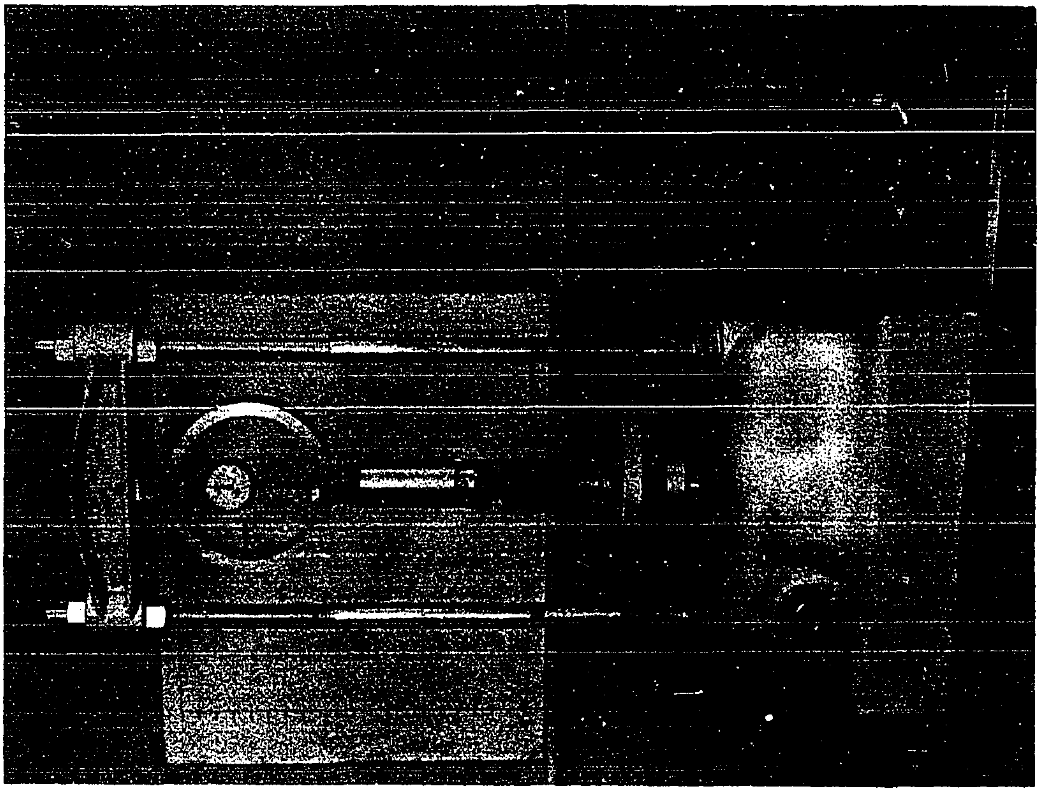
(c) EXTRUSION APPARATUS

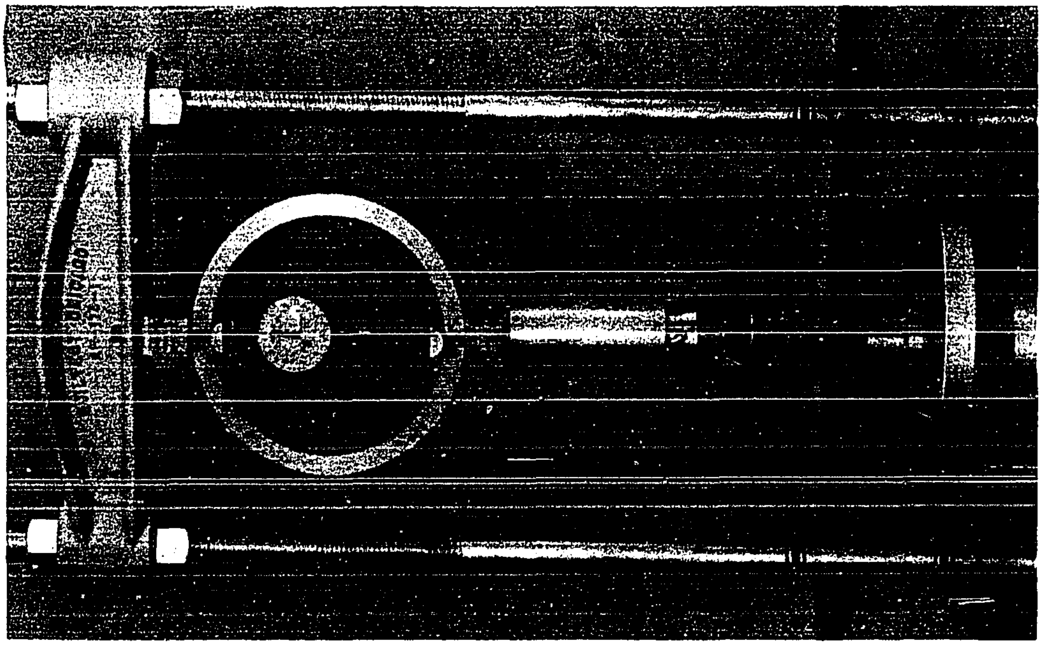


(d) HEIGHT MEASURING APPARATUS

(e) MOLDING SPECIMENS

Figure 3. Unconfined compression machine





METHODS OF EVALUATION

Preliminary Investigation

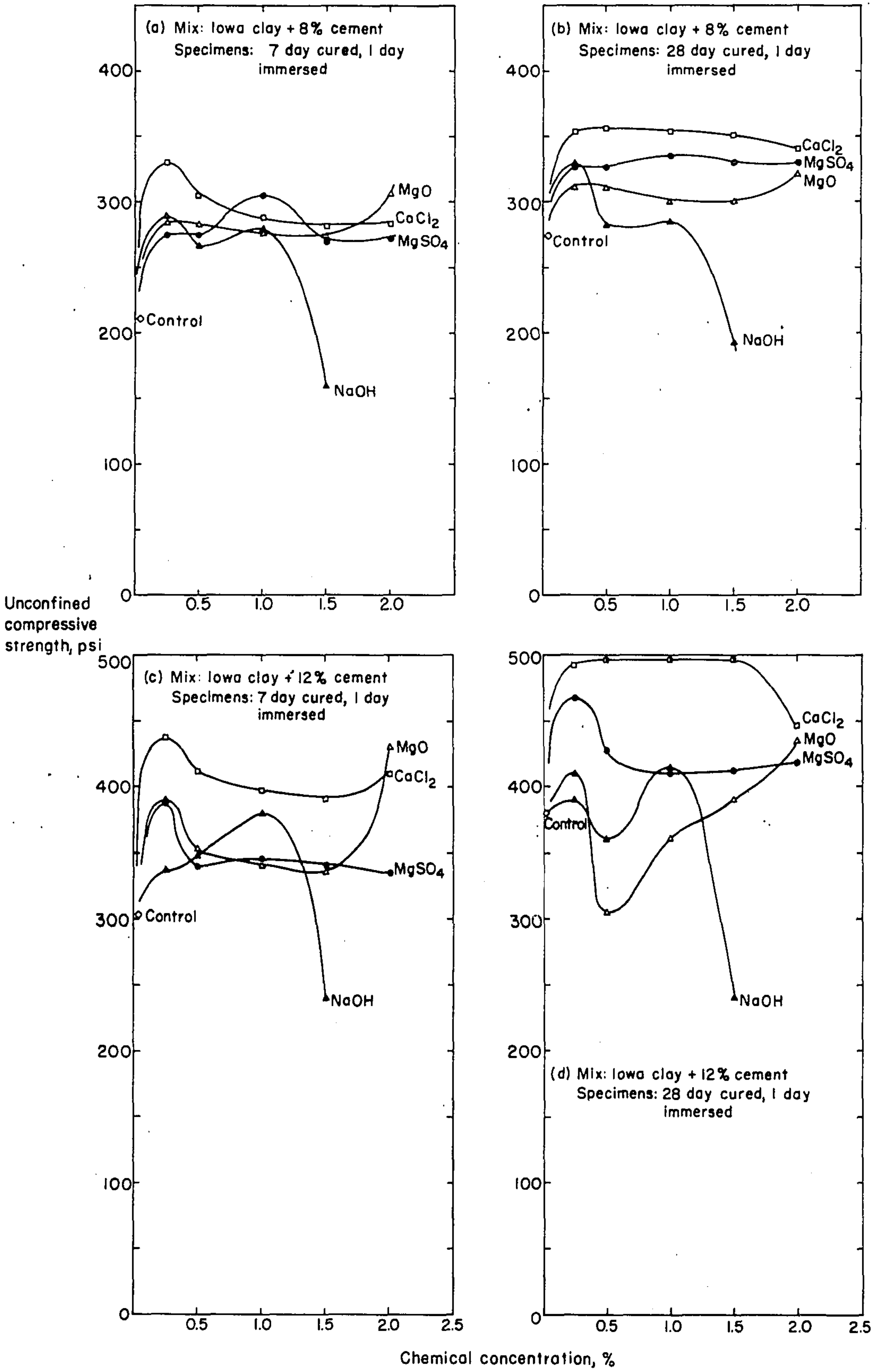
The first phase of evaluating the effectiveness of the chemicals consisted of determining the 7 day and 28 day cured, one day immersed unconfined compressive strength of soil-cement specimens to which two different amounts of a chemical had been added. The cement contents used were 4, 8, and 12 percent and the chemical contents tried were 0.0, 0.5, and 1.5 percent, all based on the oven dry weight of the soil. 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 increase of strength with time.

Judging from the strength results of the above work and in order to bracket more closely the optimum amount of chemicals for maximum strength, similar additional strength tests were run with soil-cement mixes containing a chemical in either two or more of the following amounts: 0.25, 0.75, 1.00, 1.25, and 2.00 percent. From the data obtained it was possible to choose the economically lowest optimum amount of chemical. Exceptions to this procedure were the hydrated limes used in amounts of 1 and 3 percent. In some cases, 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 and practical, too. Figure 4 is given as an example to indicate the variation of strength with chemical content and the method used in arriving at the selection of the optimum chemical content.

Hydrated lime in combination with chemicals was tried for the purpose of comparing the effectiveness of the chemical additives in soil-lime to that observed in soil-cement. Lime, as the main stabilizing agent, was tried out in amounts of 1, 3, 6 or 3, 6, 9 or 3, 5, 9 percent. The 1, 3, 6 series was used with lime A and the 3, 6, 9 or 3, 5, 9 with lime B, the selection of these percentages being based on earlier work with lime stabilization of soils (28). The evaluation procedure with soil-lime-chemical studies was similar to that with soil-cement.

Tables 12 to 19 in Appendix A present a summary of the optimum amount of chemicals found in the soil-cement and soil-lime studies.

Figure 4. Effect of varying concentration of chemical on Iowa clay stabilized with cement

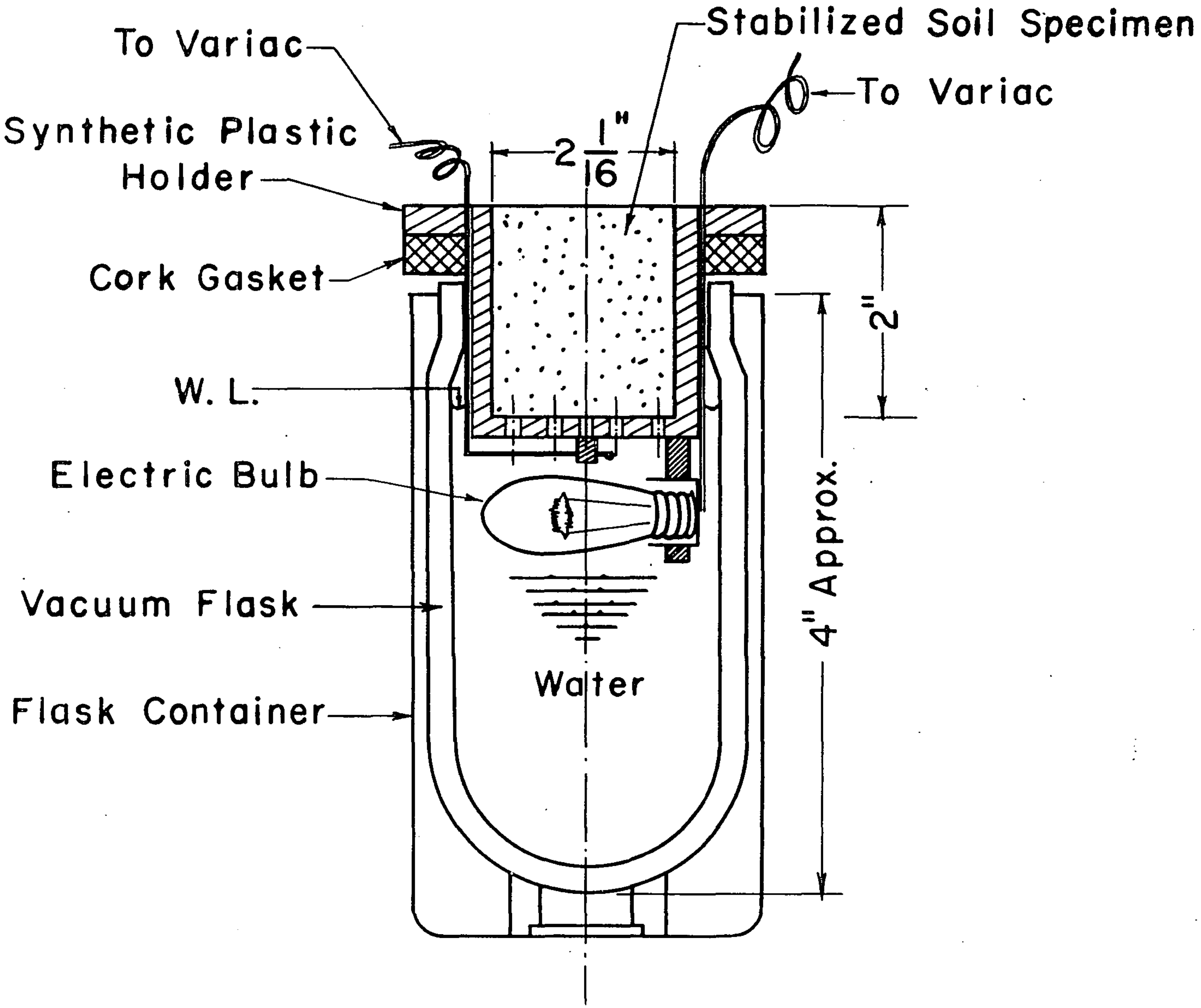

(a) Mix: Iowa clay + 8% cement
Specimens: 7 day cured, 1 day immersed
(b) Mix: Iowa clay + 8% cement
Specimens: 28 day cured, 1 day immersed
(c) Mix: Iowa clay + 12% cement
Specimens: 7 day cured, 1 day immersed
(d) Mix: Iowa clay + 12% cement
Specimens: 28 day cured, 1 day immersed
Unconfined compressive strength, psi
Chemical concentration, %
MgO
$CaCl_2$
$MgSO_4$
NaOH
Control
0
100
200
300
400
500
0.5
1.0
1.5
2.0
2.5

Tests for Durability

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

The procedure for the freeze-thaw test used in this work was developed in the Iowa Engineering Experiment Station Laboratory (19) to suit the climate in Iowa. The apparatus for this test is shown in Figure 5. Two specimens, 2 inches in diameter by 2 inches 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 received a seal coat of resin-base paint (Plax) before the specimens were immersed in water for 24 hours. One specimen was removed from the water and placed in the specimen holder and the assembly placed in a vacuum flask which contained sufficient distilled water, at a temperature of 34 ± 1.6°F, to immerse the bottom 1/4 inch 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 removed and thawed for eight hours at a temperature of 77 ± 4°F. This procedure constituted one cycle of freeze-thaw; the specimen was subjected to ten such cycles. The height of the specimen was measured at the beginning of the cycle, at

Figure 5. Iowa freeze-thaw test apparatus


To Variac
Stabilized Soil Specimen
To Variac
Synthetic Plastic Holder
$2\frac{1}{16}$"
Cork Gasket
2"
W. L.
Electric Bulb
4" Approx.
Vacuum Flask
Water
Flask Container

the end of the freezing period and at the end of the thawing period in order to calculate the amount of heave. Also, by weighing the specimen at the beginning and after the completion of the freeze-thaw cycles, it was possible to determine the amount of moisture absorbed during the test. 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 all through the duration of the freeze-thaw cycles and 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 the immersion period. The index of resistance to the effect of freezing (R_f) was calculated by the equation:

$$R_f = \left(\frac{P_f}{P_{c_f}}\right) (100) \text{ in percent}$$

where

P_f = unconfined compressive strength of freeze-thaw specimen in psi

P_{c_f} = unconfined compressive strength of control specimen in 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 seven days, then immersed for seven days. The "control" specimen 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 the formula:

$$R_i = \left(\frac{P_i}{P_{c_i}}\right) (100) \text{ in percent}$$

where

P_i = unconfined compressive strength of immersed specimen in psi

P_{c_i} = unconfined compressive strength of control specimen in psi

On the basis of the 7 day cured, one 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, A. S. T. M. Designation D560-57 (2, p. 1182).

Triaxial Compression Tests

For each soil three different mix batches were prepared. The first was that of 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 in so far as 7 day strength and resistance to freeze-thaw were concerned.

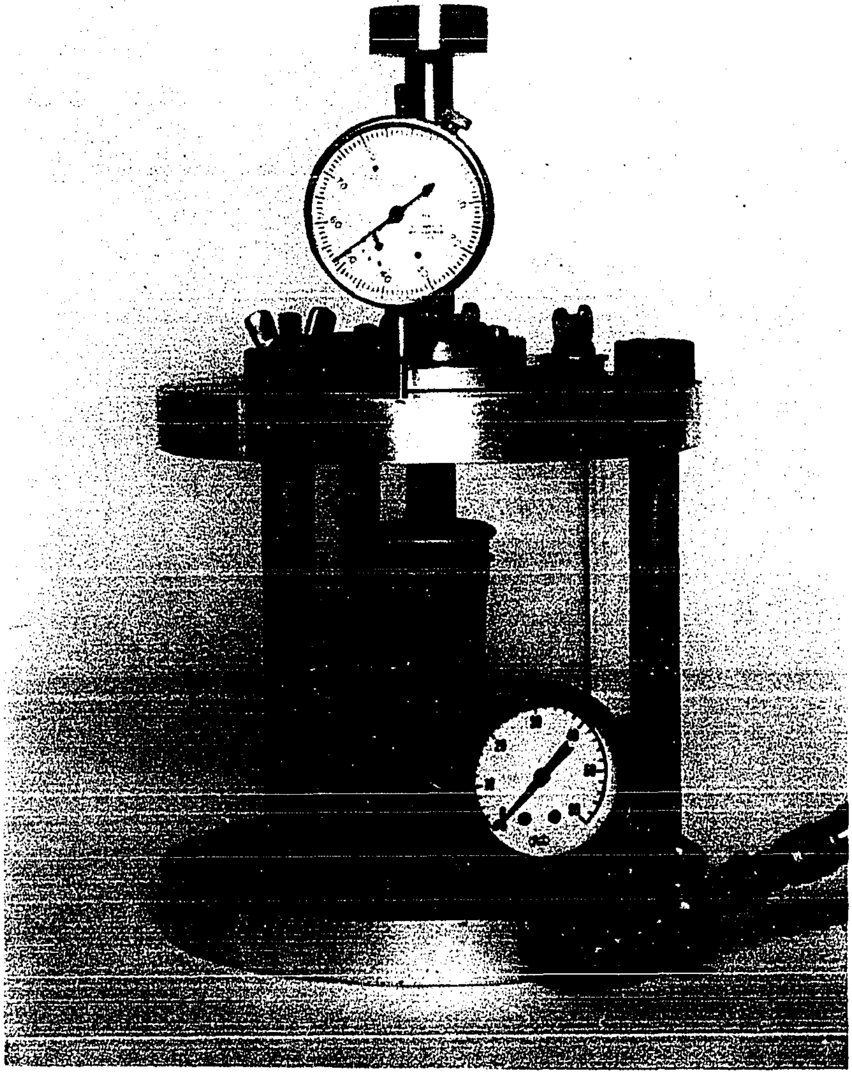
Preparation of each batch was done as explained on pages 26 to 29.

From each batch nine cylindrical specimens, 1.312 inch diameter by 2.816 inches high, were molded using the Harvard Miniature Compaction Apparatus, which gives a compacted sample at approximately standard Proctor density.

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

At the end of the curing period the cohesion and angle of internal friction of each specimen was determined by using the miniature triaxial shear testing apparatus shown in Figure 6 (24). The soil specimen, in a thin rubber membrane, was put inside the plexiglass cylinder chamber, which was well sealed against any leakage. Lateral compression, in the form of air pressure was provided by a tire pump connected to the hose at the base of the apparatus and was checked by the air pressure indicator also at the base of the apparatus. Three constant air pressures 10, 20, and 30 psi were used. At each of the three lateral pressures triplicate specimens for each combination were run and the results averaged.

Figure 6. Miniature triaxial compression apparatus



The vertical stress was applied to the specimen through the loading piston at a constant rate and both deformation and loads were recorded leading to failure.

The cohesion and angle of internal friction of any particular mix were determined graphically by means of the Mohr diagram.

PRESENTATION AND DISCUSSION OF RESULTS

In view of the relatively extensive data, the results for each soil are presented separately.

Unconfined Compressive Strength of Soil-Cement Mixtures

Iowa silt

The test results presented in Figure 11 indicate that the addition of 4 percent cement to Iowa silt produces low strength mixtures, which cannot be ameliorated by any of the chemicals incorporated in the mix. 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. While the sulfate data contradict the well-known destructive effect of sulfates on concrete and clays stabilized with cement (40), it is in agreement with reported results on sandy soils and especially with the unique effectiveness of sodium sulfate (16). 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 stabilizing effect of magnesium, sodium and calcium ions. The paradox, therefore, 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 of the paradox observed may be based on the hypothesis that the basic compounds, such as sodium hydroxide and limes, increase the solubility of organic complexes with attendant uniform distribution through the water in the mix and an interference of the formation of silica gel takes place. 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. Since between the addition of 0.5 percent and 1.5 percent calcium chloride, the former imparts relatively higher strength to the Iowa silt-cement mix, it cannot be advanced that high concentrations of calcium chloride may overcome the deleterious presence of the organic matter.

Ineffectiveness of magnesium oxide seems to be derived from a poor solubility condition. Visual 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 clay

All the chemicals and limes used with Iowa clay-cement mixes contributed to strength increase indicating that they speeded up soil-cement reaction at all cement contents used (Figure 12).

In comparison with the sulfates used, calcium chloride and magnesium oxide were very beneficial. This is in contrast to 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 partly traced to their alkalinity or acidity. The Iowa silt is well leached and highly acid with a pH value of 5.3, whereas 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 behaviour of the two soils may also accrue from the fact that one is silty and the other clayey. Although not fully elucidated for all chemicals, recent research (29) indicates that cement stabilization of friable loess does not benefit from the addition of lime; plastic loess does.

Wisconsin sand

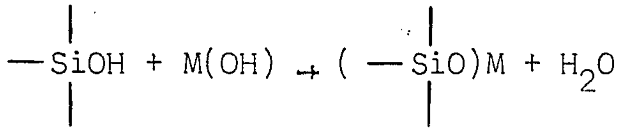
Besides being organic, the non-plastic Wisconsin sand displays uniformity of particle size and relative absence of binder material. These properties make it very difficult to stabilize as indicated in Figures 13 and 14. Cement contents up to 16 percent impart very little strength. Based on the observation that molding and extrusion of specimens were very difficult and that they seem to crumble when handled right after compaction, it may be concluded that cement failed to confer the cohesive property to the soil. With the exception of 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 were also ineffective, which 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 some 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 as shown in Figure 14. The unique effectiveness of magnesium sulfate, then, may be attributed to two factors: one, the incorporation of sulfate, and two, the small ionic radius (0.78 Å) of magnesium as compared to the large ionic radii of sodium (0.98 Å) and calcium (0.99 Å). The possible formation of a continuous skeleton, which may lead to the attainment of a useful strength level should not be disregarded.

Illinois clay

Although as low as 4 percent cement imparted a strength slightly higher than the minimum acceptable, 250 psi, the possibility of improving the Illinois clay-cement mixture was investigated by incorporating in it some chemicals and lime. The results of this investigation are presented graphically in Figure 15.

Sodium hydroxide and lime are the only additives which upgrade the quality of the Illinois clay-cement mixtures; the other chemicals investigated in this study seem to have a destructive effect. The fact that 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 bears out the thesis that the benefit derived from sodium hydroxide and lime is not due primarily to the addition of sodium and calcium ions. It seems to be due rather to the fact that 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 silica surfaces and quartz are, at least, partly covered with acidic SiOH groups (26). This reaction may be presented by



$$-\overset{|}{\underset{|}{\mathrm{Si}}}\mathrm{OH} + \mathrm{M(OH)} \rightarrow (-\overset{|}{\underset{|}{\mathrm{Si}}}\mathrm{O})\mathrm{M} + \mathrm{H_2O}$$

where 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 milliequivalents per 100 grams, 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 (44). Mixtures containing 1.5 percent sodium hydroxide were weaker than those containing 0.25 percent and in fact 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 clay

The response of the Texas clay-cement mixtures to chemical treatments is slightly different from that of Illinois clay-cement mixtures primarily because Texas clay is a heavy clay. The amount of clay-size material is higher, the predominant clay mineral is montmorillonite and 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, as shown in Figure 16, indicating a slower rate of reaction. Although lime A increased the strength of the Texas clay-cement mixtures when used in amounts of 1 percent, it gave higher strengths at 3 percent. Again the optimum amount of 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, while the carbonate proved effective only in those mixes containing 12 percent cement. The possibility that these two sodium compounds were effective with Texas clay but not with Illinois clay because of the fineness of the Texas soil leading to a more surface area available and therefore to a more extensive chemical reaction, cannot be excluded. The same reason may also be used to explain the slight benefit derived from mixing calcium chloride with the Texas clay-cement.

Michigan clay

The results in Figure 17 show that the Michigan clay-cement mixtures have an unusual affinity for the chemicals studied. Although the strength improvement derived from using some of the chemicals may be called erratic, like the benefits from calcium chloride, because of the lack of a definite trend in strength increase, for other chemicals (sodium hydroxide and carbonate) and lime, the improvement of the Michigan clay-cement mixtures is most notable. The validity of the earlier suggestion in this study 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, too. For sodium hydroxide, calcium sulfate and sodium carbonate, the optimum amounts were less than 0.5 percent and for lime 1 percent.

The addition of sodium carbonate to Michigan clay-cement mixtures appears to offer great possibilities of producing extreme stability, which results from the interaction between the calcium of cement and the sodium carbonate and the subsequent formation of a skeletal calcium carbonate in the soil mass is possible. Also, the release of sodium hydroxide acts in a way to accelerate the soil-cement reaction. There is fairly strong evidence (31) for the acceptance of the above explanation. A point of interest emerges from comparing the sodium carbonate treatment of Michigan clay-cement mixtures 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 basicity of the soils. Michigan clay is a nearly neutral soil (pH = 6.9); the other soils are either acid (pH below 6) or alkaline (pH above 8). The sodium carbonate calcium reaction seems to be most evident under neutral conditions.

North Carolina clay

Judging from the type of clay mineral, a kaolinite-halloysite intermediate, in the North Carolina clay it would be expected 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, shown in Figure 18. On the basis of the simplified explanations given for the other clay soils, it seems virtually certain that a secondary function of the optimum 3 percent lime used as an admixture, is 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 North Carolina clay-cement mixtures containing 1.5 percent sodium hydroxide and 3 percent lime is helpful in explaining the anomalous behaviour of the various sodium compounds and calcium compounds under acid conditions such as present in the North Carolina clay (pH = 5.4). As long as the environment is acidic, the precipitation of the 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 the hydroxide produces near neutral solutions which 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.

Washington sand

The high strengths attained with 4, 8 and 12 percent cement and the marked difference in 28 day strengths (Figure

19) 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 the existence of two separate reactions: cement and sand, and lime 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.

First of all, optimum conditions with the above three types of compounds were reached at 0.5 percent for all cement contents. When 1.5 percent of the chemicals were used, the strength decreased. In order to account for the increase in strength when chemicals supposedly deleterious to concrete 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.

Unconfined Compressive Strength of Soil-Lime Mixtures

In undertaking the study of determining the effect of chemicals in the soil-lime mixtures, the primary purpose was to establish, if possible, a qualitative comparative scale rather than a quantitative scale. It can be qualitative because the stabilizing effect of both cement and lime on soils is associated with the formation of cementitious calcium silicates. With regard to a possible quantitative comparison, the data may not provide the required dependability: with cement stabilization high strengths have been recorded, but with lime strengths are low to the degree that a small experimental variation may be misinterpreted. A similar error may arise when comparing 7 day strengths. Since such inaccuracies cannot be completely remedied, only the 28 day strengths of lime stabilized soils are reported in this study. They are presented graphically in Figures 20 to 27.

Lime stabilization of organic top soils does not give the minimum strength of 250 psi and although the soil-lime mixtures are benefited by the sulfates of sodium, calcium and magnesium, strengths remain below the acceptable level.

In clay soils, addition of even small amounts of lime produces a notable improvement in their workability. This is the consequence of the predominant interaction between the clay particles and the calcium ions. The effect of the chemicals, at least those which increase the strength of soil-lime mixtures, is considered marginal. The observation that with clay soils the chemicals showing a tendency to benefit soil-lime are the compounds of sodium, substantiates findings of earlier work (27). Apparently the sodium compounds promote the reaction of calcium and silica by either making more silica available for the reaction or by providing suitable alkaline conditions for the reaction to take place. Of the two sandy soils, only Washington sand responded to lime stabilization coupled with sodium compound treatment. As expected for predominantly quartz soils, sodium hydroxide increased the strength of the Washington sand-lime mixtures more than the other sodium compounds.

Durability Tests

One of the most profound observations regarding the performance of highways is the loss of strength of the base course and especially of the subgrade during springtime. There seems no doubt that the phenomenon of strength loss originates because of the presence of an excessive amount of water which has been 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 reached or tends to reach its plastic state.

The durability indices, R_f and R_i, and the unconfined compressive strengths given in Tables 20 to 44, Appendix C, reflect the susceptability of the stabilized soils to weather condition variations. 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 (45, p. 126).

Freeze-thaw tests

Figures 7 and 8 represent plots of P_{c_i} versus P_f values for the stabilized soils. Although these tables and figures give a quantitative evaluation of the resistance offered by the stabilized soil to the destructive influence of water and/or ice, the relationships established or deduced can only be interpreted and envisaged as a general tendency of the particular soil to behave as it did. No attempt was made to treat the data statistically because the experiment was not planned as such and the data seem meager for such a treatment.

Figure 7. Strength-durability variations in soils stabilized with optimum amount of chemical at varying cement contents

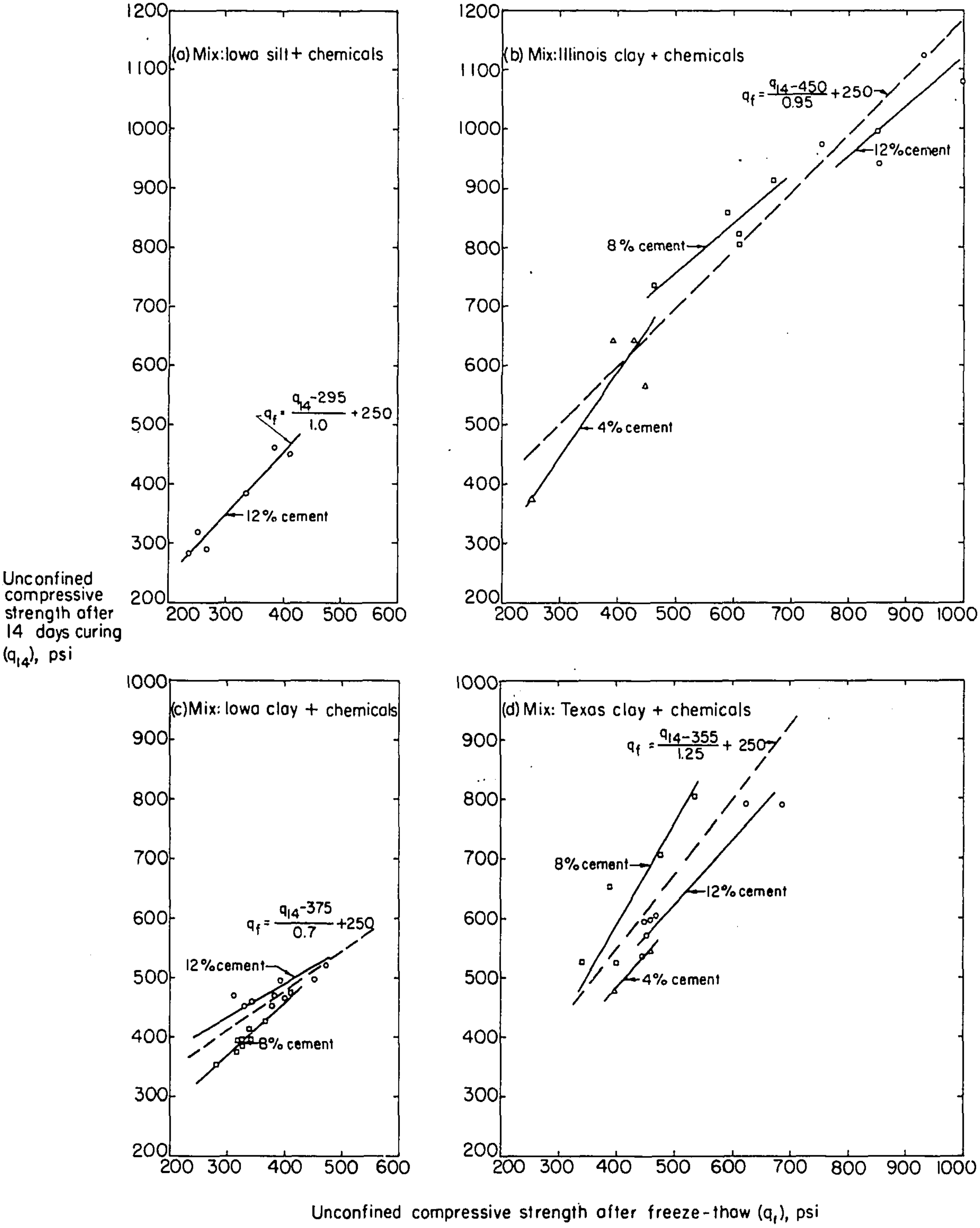
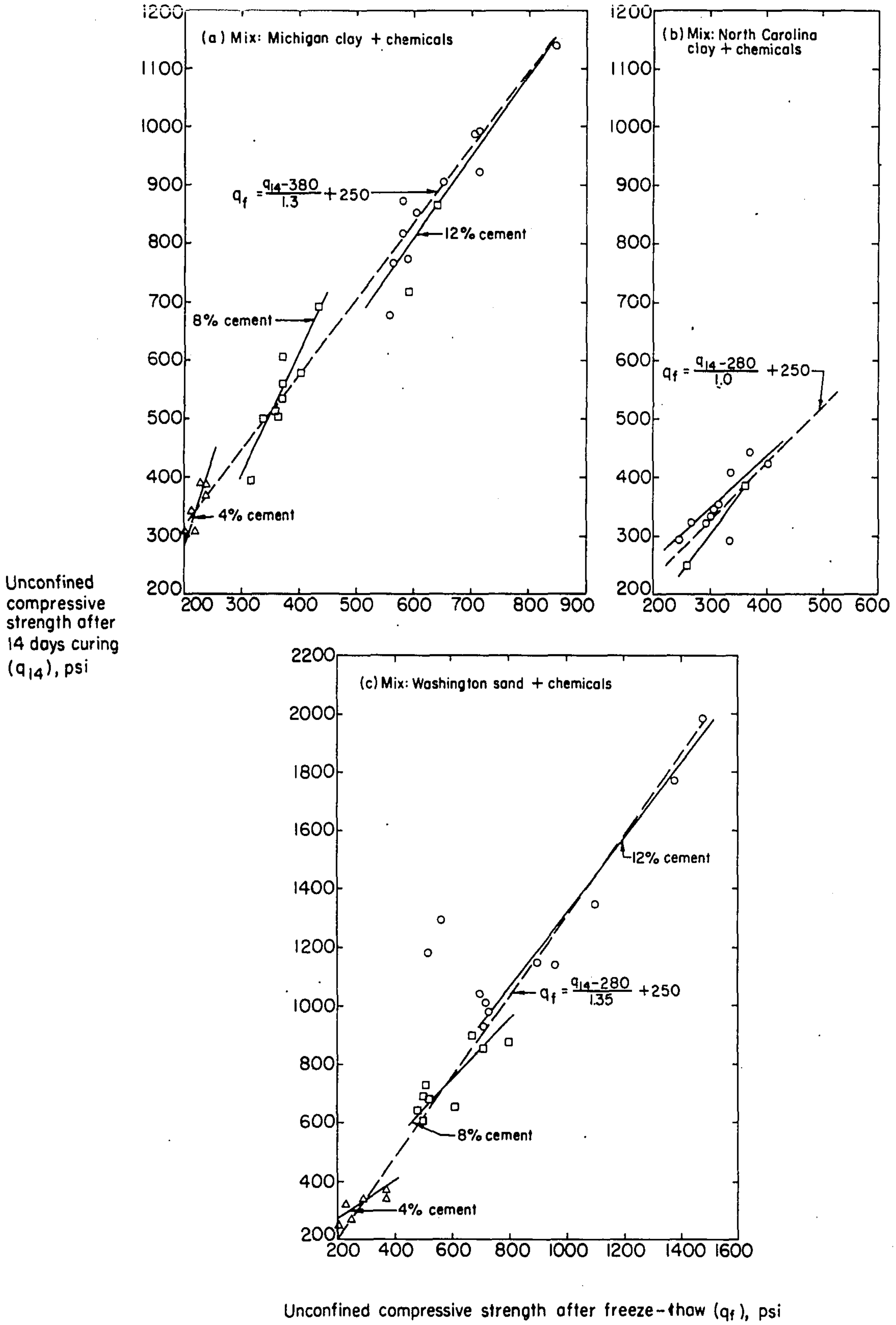

(a) Mix: Iowa silt + chemicals
$q_f = \frac{q_{14}-295}{1.0} + 250$
12% cement
(b) Mix: Illinois clay + chemicals
$q_f = \frac{q_{14}-450}{0.95} + 250$
12% cement
8% cement
4% cement
(c) Mix: Iowa clay + chemicals
$q_f = \frac{q_{14}-375}{0.7} + 250$
12% cement
8% cement
(d) Mix: Texas clay + chemicals
$q_f = \frac{q_{14}-355}{1.25} + 250$
8% cement
12% cement
4% cement
1200
1100
1000
900
800
700
600
500
400
300
200
200 300 400 500 600
200 300 400 500 600 700 800 900 1000
Unconfined compressive strength after 14 days curing (q_{14}), psi
Unconfined compressive strength after freeze-thaw (q_f), psi

Figure 8. Strength-durability variations in soils stabilized with optimum amount of chemical at varying cement contents


(a) Mix: Michigan clay + chemicals
$q_f = \frac{q_{14}-380}{1.3} + 250$
12% cement
8% cement
4% cement
(b) Mix: North Carolina clay + chemicals
$q_f = \frac{q_{14}-280}{1.0} + 250$
(c) Mix: Washington sand + chemicals
12% cement
$q_f = \frac{q_{14}-280}{1.35} + 250$
8% cement
4% cement
Unconfined compressive strength after 14 days curing (q_{14}), psi
Unconfined compressive strength after freeze-thaw (q_f), psi
200
300
400
500
600
700
800
900
1000
1100
1200
1400
1600
1800
2000
2200

From the plots of P_{c_i} versus P_f a convenient algebraic relation may be established in the form of:

$$P_f = \frac{P_{c_i} - b}{m} + 250$$

where

P_f = the unconfined compressive strength in psi of the specimen cured 7 days and subjected to ten cycles of freeze-thaw

P_{c_i} = the unconfined compressive strength in psi of the 14 day cured specimen

b = the P_{c_i} - intercept at $P_f = 250$, in psi

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 of the three cement contents used, that is, 4, 8, and 12 percent cement. On the other hand, and equally reasonable, is the establishment of a general equation, irrespective of the variations in cement content. The equation may be called the "durability conversion equation" and its importance, from the practical point of view, lies in the fact that the loss of strength expected as a result of freeze-thaw, can be obtained from the 14 day strength. Thus the same information is obtained as from the freeze-thaw test along with a simplification in equipment and technique. In order to avoid a confusion between the P_f and P_{c_i} as used in the durability tests on one hand and a 14 day strength on the other, the equation may be lettered as:

$$q_f = \frac{q_{14} - b}{m} + 250$$

where

q_f = the expected strength after freeze-thaw, in psi

q_{14} = the unconfined compressive strength in psi of the 14 day cured stabilized soil specimen

The use of these formulae presented in Table 8 may give rise to criticisms. Among these, 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 to the above criticism, it should be emphasized that the scope of establishing a relationship between freeze-thaw strength values and n day strength values is to eliminate, if possible, the freeze-thaw test and at the same time having a tool to predict 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_{c_f}, P_i or P_{c_i} values is less than the minimum 250 psi, it means that the soil is not

Table 8. Durability conversion equations, $q_f = \frac{q_{14} - b}{m} + 250$

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	$q_f = \frac{q_{14} - 490}{0.8} + 250$	
			$q_f = \frac{q_{14} - 450}{0.95} + 250$

Table 8. (Continued)

Stabilized soil type	Cement content, %	$q_f = \frac{q_{14} - b}{m} + 250$	Overall equation
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	$q_f = \frac{q_{14} - 320}{1.4} + 250$	
			$q_f = \frac{q_{14} - 380}{1.3} + 250$

Table 8. (Continued)

Stabilized soil type	Cement content, %	$q_f = \frac{q_{14} - b}{m} + 250$	Overall equation
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	$q_f = \frac{q_{14} - 330}{1.3} + 250$	
			$q_f = \frac{q_{14} - 330}{1.3} + 250$

adequately stabilized and, therefore, that particular type of mix should not be used or should be improved. In cases where the above values surpass the minimum limit of 250 psi, but the durability index is less than 80 percent, then the design value of the mix is not the least of the four values as determined experimentally; the design value to be used is the one adjusted to give a durability index of 80 percent. Usually the adjustment calls for a reduction of the experimentally determined strength value.

In some cases it was observed that some stabilized soil specimens gained strength during the freeze-thaw cycles. Although the resulting strength, P_f, was never greater than P_{c_i}, it did give freezing index values (R_f) greater than 100 percent, which means that the freeze-thaw cycles corresponded to a curing treatment. That this is true with nearly all Iowa silt specimens, but an exception with clay and sand specimens, contradicts previous results (41). 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, then, the usual freezing behavior of the stabilized Iowa silt, use is made of the basic Stefan mechanism (45, p. 131) which essentially postulates that the depth of frost penetration, z, depends not only on the thermal conductivity, K, but also on the volumetric heat of latent fusion, L, where L is 1.43

Table 9. Depth of frost penetration of stabilized soil

Stabilized soil type	Frost penetration, z, 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

times the product of dry density and moisture content of the soil mass. In this basic relation, shown by the formula:

$$z = \sqrt{\frac{48KF}{L}}$$

F is the environmental factor in degree days. By using the K values (45, p. 132) and calculating the L values for each soil, the effect of the intrinsic factors is taken into account. Disregarding the environmental effect, same for all stabilized soil specimens, the calculated depths of frost penetration, z, in the field are shown in Table 9. Iowa silt displays 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 1/4 inch, is in a humid atmosphere, which may account for the gain of strength as compared to the fully immersed specimens which reflected either loss of strength or a slower gain of 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 of which were very close to 100 percent, and one 115 percent. With Wisconsin sand, one out of three specimens gave a freezing index of 110 percent.

Clays indicate freezing indices less than 100 percent, with the exception of very few.

Therefore, while the Stefan mechanism contributes to the understanding of the behaviour of Iowa silt and Washington sand, it fails to cover the other soils, where some interfering effects, possibly of a chemical nature, might have taken place.

In pavement design, the selection of a mix is not based entirely on strength loss due to freezing. Heaving is also taken into account because its extent is compared to the permissible deformation in the pavement. At the same time, the rate of heave reflects the permeability of the soil which, in turn, 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 inch stabilized soils specimens indicate that the amount of frost heave was never greater than 0.070 inch, which corresponds to a rate of heave of less than 0.2 mm/day, a value considered "negligible" according to the U. S. Corps of Engineers frost classification. That the specimens evaluated by the Iowa freeze-thaw test may be considered as effectively stabilized is also supported by the observation that the increase in the moisture content of the specimens, usually ineluctable in freeze-thaw tests, is small.

Standard freeze-thaw tests

In comparing the results of the Iowa freeze-thaw test with those of the standard test, given in Table 10, it becomes obvious that those mixes which passed the standard test were considered successfully stabilized on the basis of the Iowa freeze-thaw test criteria. Exception to this is 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.

It is unlikely that the results obtained from the two tests can be correlated because of the different nature of the two tests. Nevertheless, the following observation is interesting. A few unconfined compressive strength tests were run on specimens indicated as No. 1 of the standard test at the end of that cycle at which failure was considered as having occurred or at the end of the 12th cycle when the mix tested showed that it withstood 12 cycles of freeze-thaw. The reason for using specimen No. 1 was that it is the specimen which is not brushed and which is used for volume change calculations. The unconfined compressive strength determined on specimens No. 1 of the standard test were very low in comparison to the P_f values of the Iowa freeze-thaw test. This shows that perhaps the stabilized soil mixes benefited more, or lost less strength, by the curing nature of the Iowa freeze-thaw test than they did by the standard test. 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 at the completion of 12 cycles of the standard test is surprising. The explanation given before regarding the depth of frost penetration may in a way account for its unusual behavior.

Table 10. Summary of standard freeze-thaw tests

Specimen	Density before freeze-thaw, pcf	Moisture before freeze-thaw, %	Number of cycles[a] completed	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

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[a] See reference 38, p. 34.

Table 10. (Continued)

Specimen	Density before freeze-thaw, pcf	Moisture before freeze-thaw, %	Number of cycles[a] completed	Conditions at completion of cycles indicated			
				Density, pcf	Moisture, %	Volume change, %	Stabilized soil loss, %
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_2CO_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_4SiO_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

Since the data collected from the Iowa freeze-thaw test indicate that, in general, those stabilized mixes which displayed an unconfined compressive strength of 250 psi or more at the end of a 7 day humid curing period performed successfully under the conditions imposed by the Iowa freeze-thaw test, the original hypothesis that the 7 day strength is dependable becomes valid.

Wet-dry tests

The results of the wet-dry tests for the stabilized soil specimens, shown in Tables 20 to 44, Appendix C, may be used as an 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 when R_i values less than 80 percent were obtained, 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 measured after freeze-thaw testing. Therefore, the use of the wet-dry test results leads to an underdesign, whereas a design based on the freeze-thaw test results will be safe. It is striking again that the Iowa silt stabilized specimens 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 as explained earlier.

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

Triaxial Compression Tests

Although much has been written about the shear resistance of soils, the mechanism of shear failure is not well understood possibly because a precise concept of failure has not been developed yet for soils. In this study, failure is accepted 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 by an exterior load.

Since the object of this phase of the investigation was to determine the relative values of the two shear components, cohesion and angle of internal friction, it seemed unlikely that the measurement of pore water pressure would be of major importance. Therefore, the values of cohesion and angle of internal friction determined from the Mohr diagrams (Figures

Table 11. Shear components of raw and stabilized soils

Sample designation	Cohesion c, psi	Angle of internal 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	$s = 50 + 0.84p$
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	$s = 30 + 1.07p$

Table 11. (Continued)

Sample designation	Cohesion c, psi	Angle of internal friction ϕ, degrees	Shear equation $s = c + p \tan \phi$
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_2CO_3	110	41	$s = 110 + 0.87p$
North Carolina clay	25	20	$s = 25 + 0.36p$
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	$s = 35 + 2.14p$

28 to 35, Appendix D) and reported in Table 11 are not the "effective" values; they are the values attained by the soil in a compacted form equivalent to that of its field condition.

Sandy soils

The two sandy soils, the Wisconsin sand and the Washington sand, have a very low cohesive strength of 5 psi in their unmodified-by-stabilizing-agents condition. Also, the Washington sand has a 35° angle of internal friction, as expected; but the Wisconsin sand exhibits an unexpected low angle of internal friction of 17°. The angular variation found between two like soils may be related to the composition of two soils. The clay content in the Wisconsin sand is higher by four percentage points; this coupled with a higher molding moisture produced a lubricant matrix, a condition not established in the Washington sand. The addition of cement has the same over all effect on both soils. It increased both the cohesion and angle of internal friction, and, it seemed to have a greater influence in improving the coarser Washington sand than the finer Wisconsin sand. Apparently cement fills in a large part of the voids in the Washington sand, thus producing a dense structure where the individual particles are not only more firmly embedded in a finer cementitious material but also their relative movement is more restricted.

The addition of a chemical, 2 percent magnesium sulfate for the Wisconsin sand stabilized with 12 percent cement, and 0.5 percent sodium hydroxide for the Washington sand stabilized with 12 percent cement, which proved beneficial on the basis of the unconfined compressive strength, 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 apparently in combination with cement produced a greater amount of gel which either encases the granular particles or acts as void filler. This has a two-fold interpretation: one, the extent of cementation between particles is greater; two, it tends to provide the soil mass with agglomerated finer particles. The first phenomenon is conducive to greater cohesion, the second to larger values of internal friction.

Clay soils

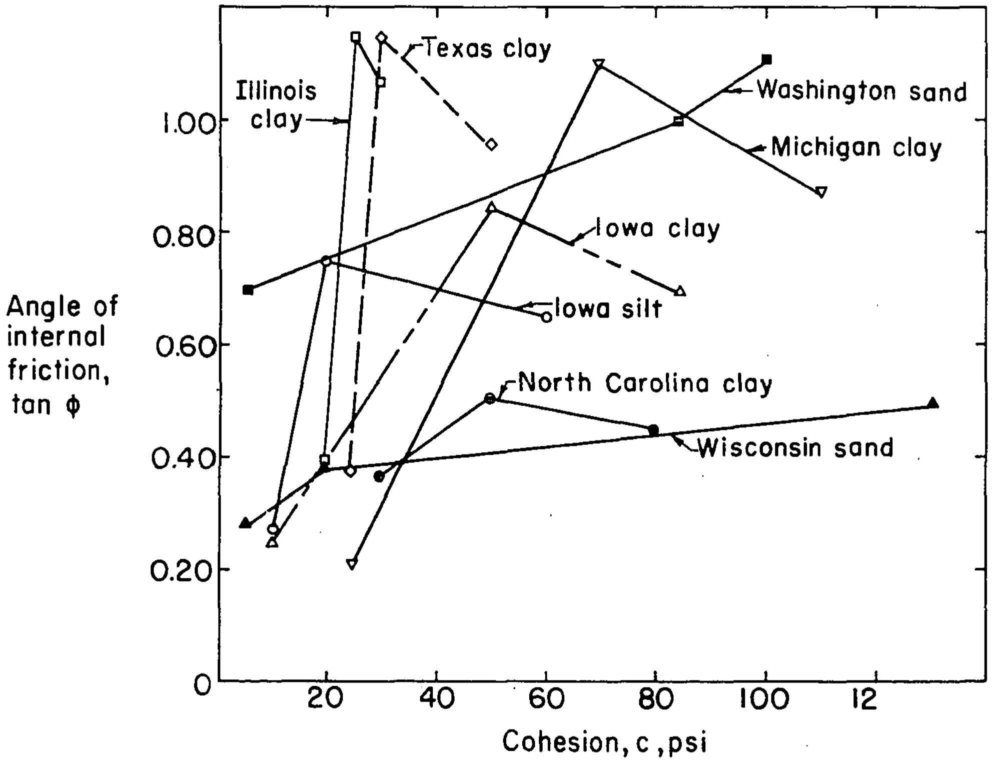
All other soils in this study have been grouped for convenience into the "clay" category although they are not classified as such texturally.

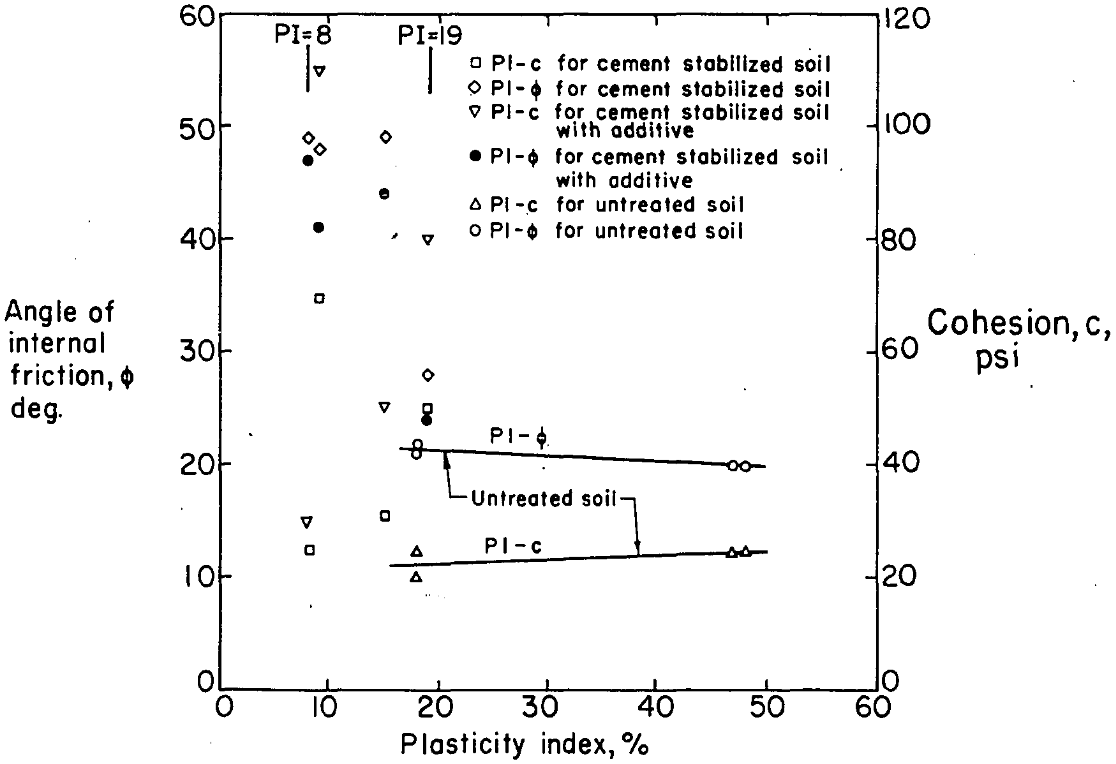
The addition of cement to clay soils seemed to have the same effect as it did to sandy soils. Their cohesive strength improved simultaneously with an increase in the angle of internal friction. The amount of increase varies from soil to soil but no definite relationship could be established between the soil type and the change in the values of the two shearing components, which are presented in Table 11. However, when the four inorganic clay soils, Illinois clay, Texas clay, Michigan clay, and North Carolina clay are considered, the slightly inclined PI-ø and PI-c curves of the untreated soil, shown in Figure 9, become nearly vertical lines when depicting the same soils stabilized with 12 percent cement. In fact, the lines fall in a band limited by PI = 8 percent and PI = 19 percent. A decrease in the plasticity index is a manifestation of aggregation, which as explained previously leads to larger internal friction angle. The cementation accruing from the presence of the cement gel was the factor that led to a higher cohesive strength.

The first and most emminent observation of the effect of chemical additive on soil-cement when the soil is non-granular, is the tendency of the mix to gain in shearing resistance; yet, as Table 11 and Figure 10 indicate, the increase in the cohesive strength is masked somewhat by a slight decrease in the angle of internal friction. That cohesion increases is not surprising based on what has been discussed so far. The decrease in the internal friction could not be traced to a possible reduced agglomeration of particles, at least, not as far as the plastic properties

Figure 9. Effect of stabilization on the cohesion and angle of internal friction of soils

Figure 10. Variation of plasticity index with cohesion and angle of internal friction of untreated and stabilized soils


Texas clay
Illinois clay
Washington sand
Michigan clay
Iowa clay
Iowa silt
North Carolina clay
Wisconsin sand
Angle of internal friction, tan ϕ
1.00
0.80
0.60
0.40
0.20
0
20
40
60
80
100
12
Cohesion, c, psi


PI=8
PI=19
□ PI-c for cement stabilized soil
◇ PI-ϕ for cement stabilized soil
▽ PI-c for cement stabilized soil with additive
● PI-ϕ for cement stabilized soil with additive
△ PI-c for untreated soil
○ PI-ϕ for untreated soil
PI-ϕ
Untreated soil
PI-c
Angle of internal friction, ϕ deg.
Cohesion, c, psi
60
50
40
30
20
10
0
120
100
80
60
40
20
0
0
10
20
30
40
50
60
Plasticity index, %

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 presence of the chemical and the following hypothesis is offered as 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 depicted in Figure 9. But the ions, which are supplied to the soil-cement mass in the form of a chemical, do take their place between already aggregated clay particles with the result that the spacing between the particles is slightly increased. Accommodating the hydrated ion means that the interparticle contact is numerically reduced. Since internal friction arises from interparticle proximity, it may be reasonable to accept the validity of the hypothesis proposed.

SUMMARY AND CONCLUSIONS

The effects of sodium, calcium and magnesium ions added to soil-cement and soil-lime mixtures in the form of the hydroxide, chloride, sulfate, orthosilicate and oxide were studied and quantitatively measured by determining the unconfined compressive strengths of 2 inch diameter by 2 inch high specimens of the mixtures at the end of a 7 day humid curing period. Using a strength criterion of 250 psi for the 7 day strength, it was possible to single out the promising chemicals and bracket their optimum content for a particular soil-cement or soil-lime mixture. The strengths obtained with soil-cement mixtures containing chemicals were compared with those obtained when commercially produced hydrated high calcium and dolomitic lime were used with soil-cement mixtures.

Of the eight soils used in this study, two were sandy, one was silty, and five were clayey. Three were organic top soils and five were B- or C-horizon soils. The clay mineral in the soils varied from kaolinite to illite to montmorillonite.

Although the following conclusions apply specifically to the 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 two 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 depends primarily on existing local conditions.

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

3. The organic top soils which are low in clay content and not alkaline, and the Iowa silt, the Wisconsin sand and the 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 limes A and B 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; these additives increased the strength of the Iowa clay-cement mixtures 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,

which are heavy clays texturally, were significantly upgraded strengthwise by additives of limes A and B or sodium hydroxide. Sodium orthosilicate seemed to accelerate the soil-cement reaction in mixtures with all soils except in the Illinois clay. The beneficial effect of sodium carbonate was limited to mixtures with Texas clay and Michigan clay.

6. Lime stabilization of the eight soils offers little promise for the use of chemicals. The Texas clay + lime A + sodium orthosilicate, the Texas clay + lime B + sodium orthosilicate or carbonate, the Michigan clay + lime B + sodium orthosilicate or calcium sulfate, the North Carolina clay + lime B + sodium orthosilicate or calcium sulfate, and the Washington sand + lime A or lime B + sodium hydroxide, were the mixtures which indicated improvement in strength.

The above 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, it can safely be proposed that:

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 for the standard freeze-thaw test because of its relative simplicity.

2. The establishment of a functional relationship between the 14 day unconfined compressive strength of the soil-cement-chemical mixture 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.

While modern theory and instrumentation have traced the stability of a soil mass to very complicated phenomena, still the outstanding valuable characteristics of a soil as a unit are its cohesion and angle of internal friction. These two properties determine the shearing strength of a soil. Triaxial shear tests run on samples, compacted to near standard Proctor density at optimum moisture content, indicated that:

1. The addition of cement increased the cohesion and internal friction angle of the Wisconsin sand and Washington sand. Magnesium sulfate for the former and sodium hydroxide for 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. When the chemicals or lime (calcium sulfate with Iowa silt, lime B with Iowa clay, sodium hydroxide with Illinois clay, lime B with Texas clay, sodium carbonate with Michigan clay, sodium hydroxide with North Carolina clay) giving optimum conditions were added to soil-cement mixtures, cohesion was substantially increased but the angle of internal friction decreased slightly; however, the net result was an overall increase of shearing strength.

FUTURE RESEARCH

The knowledge gained from the present study was helpful in formulating explanations which, because of their speculative character, should be regarded as tentative till further observations substantiate them. The type of test experiments which will be required along this line may be divided into two broad areas:

1. Determination of the effect of chemicals on either cement or lime stabilization of soils by employing pore water measurements in the triaxial compressive strength tests. Such a program of testing will provide more precise information on the effective values of shear components. This is important in view of the recent interest which has developed in connection with pore water pressures and hydraulic or capillary conductivity of soils.

2. The determination of either physical or chemical changes that have occurred in the soil, as a result of the stabilization procedure, by means of X-ray diffraction analysis. Since the primary effects of the chemicals are almost centered in the clay fraction, the basal d-spacing and line broadening measurements will possibly throw some light on the changes of the internal structure of the soil, if any.

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APPENDIX A

Table 12. Optimum additive content[a] for the Iowa silt, percent of oven dry weight of soil at various cement and lime contents

Additive	Cement, %			Hydrated lime, %					
				A			B		
	4	8	12	1	3	6	1	3	6
Sodium hydroxide	0.50	0.50	0.50	--	0.50	0.50	--	--	0.50
Calcium sulfate	0.50	1.50	2.00[b]	0.50	0.50	0.50	--	1.50	1.50
Magnesium sulfate	0.50	0.50	2.00[b]	0.50	0.50	0.50	--	0.50	0.50
Sodium sulfate	0.50	1.00	1.00[b]	--	0.50	0.50	--	0.50	0.50
Calcium chloride	0.50	0.50	0.50[b]	--	0.50	0.50	--	0.50	0.50
Magnesium oxide	0.50	0.50	0.50	--	0.50	0.50	--	0.50	0.50
Sodium orthosilicate	--	--	--	--	--	--	--	--	--
Sodium carbonate	--	--	--	--	--	--	--	--	--
Lime A[c]	1.00	1.00	1.00	--	--	--	--	--	--
Lime B[c]	1.00	1.00	1.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 13. Optimum additive content[a] for the Iowa clay, percent of oven dry weight of soil at various cement and lime contents

Additive	Cement, %			Hydrated lime, %					
				A			B		
	4	8	12	1	3	6	1	5	9
Sodium hydroxide	0.50	0.25[b]	1.00[b]	1.50	0.50	0.50	--	0.50	0.50
Calcium sulfate	0.50	0.25[b]	0.25[b]	1.50	0.50	0.50	0.50	0.50	0.50
Magnesium sulfate	0.50	1.00[b]	0.25[b]	--	0.50	0.50	0.50	0.50	0.50
Sodium sulfate	0.50	0.50[b]	0.50[b]	--	0.50	0.50	0.50	0.50	0.50
Calcium chloride	1.50	0.25[b]	0.25[b]	0.50	0.50	1.50	0.50	0.50	0.50
Magnesium oxide	0.50	2.00[b]	2.00[b]	1.50	1.50	1.50	0.50	0.50	0.50
Sodium orthosilicate	--	--	--	--	--	--	--	--	--
Sodium carbonate	--	--	--	--	--	--	--	--	--
Lime A[c]	3.00	3.00	1.00	--	--	--	--	--	--
Lime B[c]	3.00	3.00	1.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 14. Optimum additive content[a] for the Wisconsin sand, percent of oven dry weight of soil at various cement and lime contents

				Hydrated lime, %					
Additive	Cement, %			A			B		
	8	12	16	1	3	6	3	5	9
Sodium hydroxide	--	0.25	--	--	--	0.25	--	--	--
Calcium sulfate	--	--	--	--	--	--	--	--	--
Magnesium sulfate	2.00	2.00[b]	2.00[b]	--	--	1.50	--	1.50	1.50
Sodium sulfate	--	--	--	--	--	--	--	--	--
Calcium chloride	2.00	2.00	2.00	--	2.00	2.00	--	2.00	2.00
Magnesium oxide	--	--	--	--	--	--	--	--	--
Sodium orthosilicate	0.50	0.50	--	--	0.50	0.50	--	0.50	0.50
Sodium carbonate	0.25	0.25	--	--	0.25	0.25	--	0.25	0.25
Lime A[c]	3.00	3.00	--	--	--	--	--	--	--
Lime B[c]	3.00	3.00	--	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 15. Optimum additive content[a] for the Illinois clay, percent of oven dry weight of soil at various cement and lime contents

				Hydrated lime, %					
Additive	Cement, %			A			B		
	4	8	12	1	3	6	3	6	9
Sodium hydroxide	0.25[b]	0.25[b]	0.25[b]	--	0.25	0.25	0.25	0.25	0.25
Calcium sulfate	1.50	0.50	0.50	--	--	--	--	--	--
Magnesium sulfate	0.50	0.50	0.50	--	2.00	2.00	2.00	2.00	2.00
Sodium sulfate	0.50	0.50	0.50	--	--	--	--	--	--
Calcium chloride	0.25[b]	0.25[b]	0.25[b]	--	2.00	2.00	--	2.00	2.00
Magnesium oxide	0.25[b]	0.25[b]	0.25[b]	--	--	--	--	--	--
Sodium orthosilicate	0.50	0.50	0.50	--	--	0.50	--	0.50	0.50
Sodium carbonate	0.25[b]	0.25[b]	0.25[b]	--	0.25[b]	0.25[b]	0.25[b]	0.25[b]	0.25[b]
Lime A[c]	1.00	1.00	1.00	--	--	--	--	--	--
Lime B[c]	3.00	3.00	3.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 16. Optimum additive content[a] for the Texas clay, percent of oven dry weight of soil at various cement and lime contents

				Hydrated lime, %					
Additive	Cement, %			A			B		
	4	8	12	1	3	6	3	5	9
Sodium hydroxide	0.25	0.25[b]	0.25[b]	--	0.25	0.25	--	0.25	0.25
Calcium sulfate	1.50	1.50	1.50	--	--	--	--	--	--
Magnesium sulfate	1.50	1.50	1.50	--	2.00	2.00	--	2.00	2.00
Sodium sulfate	0.50	0.50	0.50	--	0.50	0.50	--	0.50	0.50
Calcium chloride	0.50	0.50	0.50[b]	--	0.50	0.50	--	0.50	0.50[b]
Magnesium oxide	1.50	1.50	1.50	--	--	--	--	--	--
Sodium orthosilicate	0.50	0.50[b]	0.50[b]	--	0.50	0.50	--	0.50	0.50
Sodium carbonate	0.25	0.25	0.25[b]	--	0.25	0.25	--	0.25	0.25
Lime A[c]	3.00	3.00	3.00	--	--	--	--	--	--
Lime B[c]	3.00	3.00	3.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 17. Optimum additive content[a] for the Michigan clay, percent of oven dry weight of soil at various cement and lime contents

Additive	Cement, %			Hydrated lime, %					
				A			B		
	4	8	12	1	3	6	3	6	9
Sodium hydroxide	0.50	0.50[b]	0.50[b]	--	--	0.25	--	0.25	0.25
Calcium sulfate	0.50	0.50[b]	0.50[b]	--	--	0.50	--	--	0.50
Magnesium sulfate	2.00	2.00[b]	2.00[b]	--	--	2.00	--	--	2.00
Sodium sulfate	0.50[b]	0.50[b]	0.50	--	--	0.50	--	0.50	0.50
Calcium chloride	0.50[b]	0.50[b]	0.50[b]	--	--	0.50	--	0.50	0.50
Magnesium oxide	1.50[b]	1.50[b]	1.50[b]	--	--	0.50	--	--	0.50
Sodium orthosilicate	0.50[b]	0.50[b]	0.50[b]	--	--	0.50	--	0.50	0.50
Sodium carbonate	0.25[b]	0.25[b]	0.25[b]	--	--	0.25	--	0.25	0.25
Lime A[c]	1.00	1.00	1.00	--	--	--	--	--	--
Lime B[c]	1.00	1.00	1.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 18. Optimum additive content[a] for the North Carolina clay, percent of oven dry weight of soil at various cement and lime contents

Additive	Cement, %			Hydrated lime, %					
				A			B		
	4	8	12	1	3	6	3	6	9
Sodium hydroxide	1.50	1.50[b]	1.50[b]	--	--	1.50	--	--	0.50
Calcium sulfate	0.50	0.50	0.50	--	--	0.50	--	--	1.50[b]
Magnesium sulfate	2.00	2.00	2.00	--	--	2.00	--	--	--
Sodium sulfate	0.50	0.50	0.50[b]	--	--	0.50	--	--	0.50
Calcium chloride	1.00	1.00	1.00	--	--	1.00	--	--	1.00
Magnesium oxide	0.50	0.50	0.50[b]	--	--	1.50	--	--	0.50
Sodium orthosilicate	0.50	0.50	0.50[b]	--	--	0.50	--	--	1.50
Sodium carbonate	1.50	1.50	1.50[b]	--	--	0.25	--	--	0.25
Lime A[c]	3.00	3.00	3.00	--	--	--	--	--	--
Lime B[c]	3.00	3.00	3.00	--	--	--	--	--	--

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

Table 19. Optimum additive content[a] for the Washington sand, percent of oven dry weight of soil at various cement and lime contents

Additive	Cement, %			Hydrated lime, %					
				A			B		
	4	8	12	1	3	6	3	6	9
Sodium hydroxide	0.50[b]	0.50[b]	0.50[b]	--	0.50[b]	0.50[b]	--	0.50[b]	0.50[b]
Calcium sulfate	0.50[b]	0.50[b]	0.50[b]	--	--	0.50	--	0.50	0.50
Magnesium sulfate	0.50	0.50	0.50[b]	--	--	0.50	--	0.50	0.50
Sodium sulfate	0.50	0.50[b]	0.50[b]	--	0.50	0.50	--	0.50	0.50
Calcium chloride	0.50	0.50	0.50[b]	--	--	0.50	--	1.50	1.50
Magnesium oxide	0.50	0.50	0.50[b]	--	--	0.50	--	0.50	0.50
Sodium orthosilicate	0.50[b]	0.50[b]	0.50[b]	--	--	0.50	--	0.50	0.50[b]
Sodium carbonate	0.50[b]	0.50[b]	0.50[b]	--	0.50	0.50[b]	--	0.50	0.50
Lime A[c]	3.00	3.00	3.00	--	--	--	--	--	--
Lime B[c]	3.00	3.00	3.00	--	--	--	--	--	--

---

[a] Values with no superscript indicate the optimum selected from two percentages: 0.50 and 1.50.

[b] Determined graphically by using four different percentages.

[c] Optimum lime content used with cement selected from two percentages: 1.00 and 3.00.

APPENDIX B

Figure 11. Unconfined compressive strengths of Iowa silt stabilized with optimum amount of chemical or hydrated lime at varying cement contents

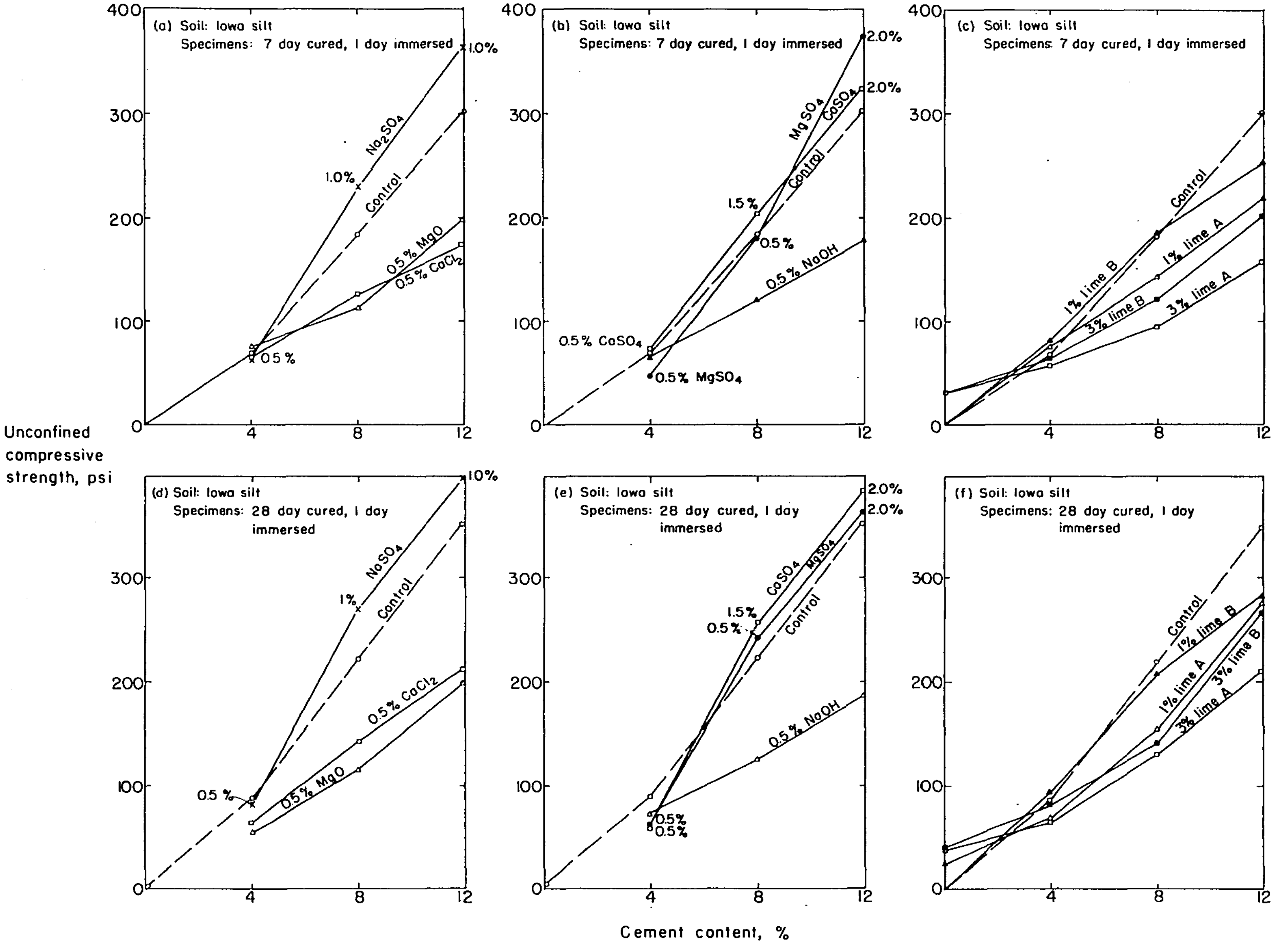

Unconfined compressive strength, psi
Cement content, %
(a) Soil: Iowa silt
Specimens: 7 day cured, 1 day immersed
Na2SO4
1.0%
1.0%
Control
0.5% MgO
0.5% CaCl2
0.5%
(b) Soil: Iowa silt
Specimens: 7 day cured, 1 day immersed
2.0%
2.0%
MgSO4
CaSO4
Control
1.5%
0.5%
0.5% NaOH
0.5% CaSO4
0.5% MgSO4
(c) Soil: Iowa silt
Specimens: 7 day cured, 1 day immersed
Control
1% lime A
1% lime B
3% lime B
3% lime A
(d) Soil: Iowa silt
Specimens: 28 day cured, 1 day immersed
10%
NaSO4
1%
Control
0.5%
0.5% CaCl2
0.5% MgO
(e) Soil: Iowa silt
Specimens: 28 day cured, 1 day immersed
2.0%
2.0%
CaSO4
MgSO4
1.5%
0.5%
Control
0.5% NaOH
0.5%
0.5%
(f) Soil: Iowa silt
Specimens: 28 day cured, 1 day immersed
Control
1% lime B
1% lime A
3% lime B
3% lime A
0
100
200
300
400
4
8
12

Figure 12. Unconfined compressive strengths of Iowa clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents

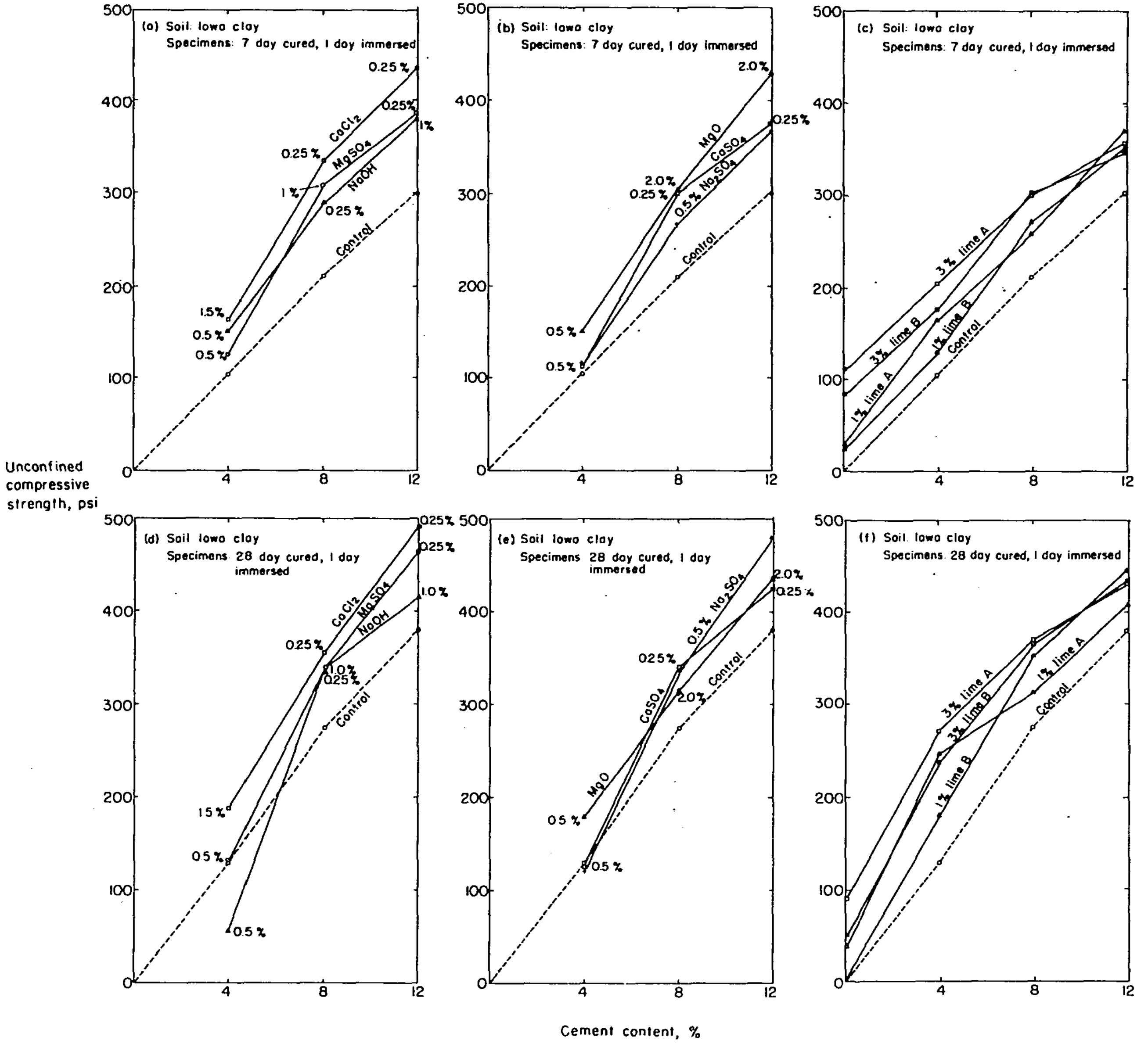

Unconfined compressive strength, psi
Cement content, %
(a) Soil: Iowa clay
Specimens: 7 day cured, 1 day immersed
CaCl$_2$
MgSO$_4$
NaOH
Control
1.5%
0.5%
0.5%
0.25%
1%
0.25%
0.25%
0.25%
1%
(b) Soil: Iowa clay
Specimens: 7 day cured, 1 day immersed
MgO
CaSO$_4$
Na$_2$SO$_4$
Control
0.5%
0.5%
2.0%
0.25%
0.5%
2.0%
0.25%
(c) Soil: Iowa clay
Specimens: 7 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
(d) Soil Iowa clay
Specimens 28 day cured, 1 day immersed
CaCl$_2$
MgSO$_4$
NaOH
Control
1.5%
0.5%
0.5%
0.25%
1.0%
0.25%
0.25%
0.25%
1.0%
(e) Soil Iowa clay
Specimens 28 day cured, 1 day immersed
MgO
CaSO$_4$
0.5% Na$_2$SO$_4$
Control
0.5%
0.5%
0.25%
2.0%
2.0%
0.25%
(f) Soil Iowa clay
Specimens 28 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
0
100
200
300
400
500
4
8
12

Figure 13. Unconfined compressive strengths of Wisconsin sand stabilized with optimum amount of chemical or hydrated lime at varying cement contents

Figure 14. Effect of $MgSO_4$ concentration on the strength of Wisconsin sand stabilized with cement

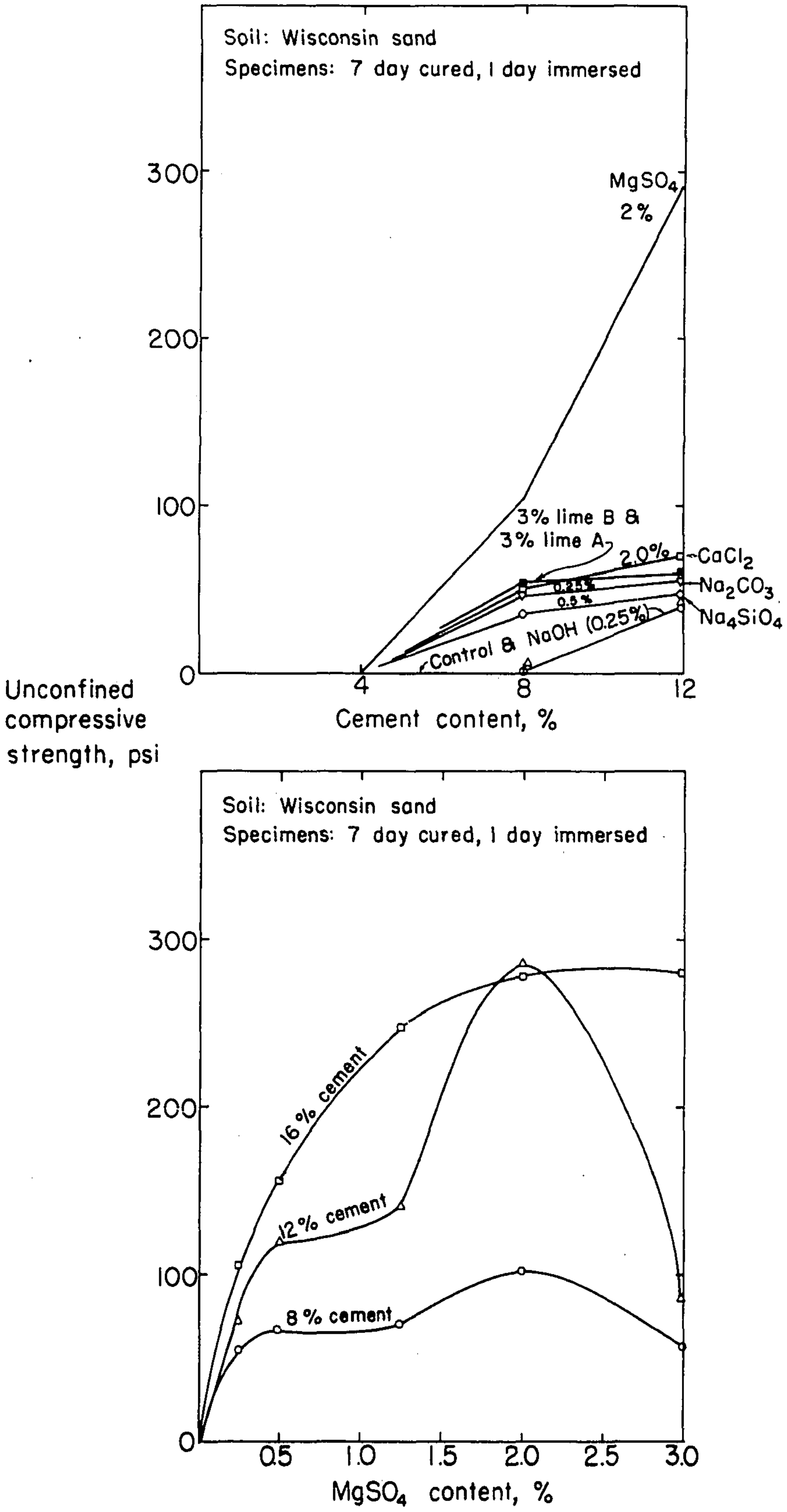

Soil: Wisconsin sand
Specimens: 7 day cured, 1 day immersed
MgSO$_4$ 2%
3% lime B &
3% lime A
2.0%
CaCl$_2$
0.25%
Na$_2$CO$_3$
0.5%
Na$_4$SiO$_4$
Control & NaOH (0.25%)
0
100
200
300
4
8
12
Cement content, %
Unconfined compressive strength, psi
Soil: Wisconsin sand
Specimens: 7 day cured, 1 day immersed
16% cement
12% cement
8% cement
0
100
200
300
0.5
1.0
1.5
2.0
2.5
3.0
MgSO$_4$ content, %

Figure 15. Unconfined compressive strengths of Illinois clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents

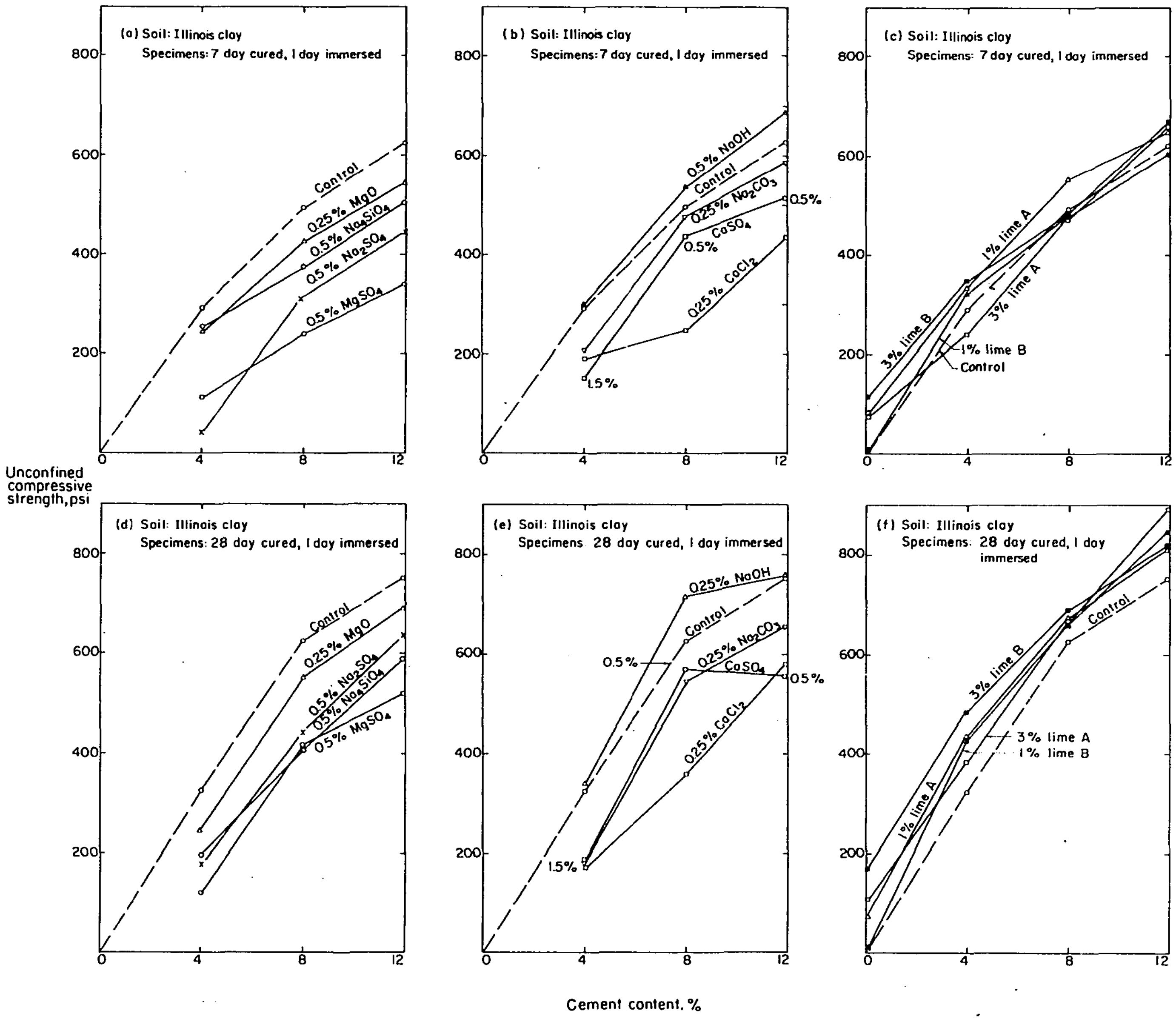

(a) Soil: Illinois clay
Specimens: 7 day cured, 1 day immersed
Control
0.25% MgO
0.5% Na_4SiO_4
0.5% Na_2SO_4
0.5% $MgSO_4$
(b) Soil: Illinois clay
Specimens: 7 day cured, 1 day immersed
0.5% NaOH
Control
0.25% Na_2CO_3
0.5%
$CaSO_4$
0.5%
0.25% $CaCl_2$
1.5%
(c) Soil: Illinois clay
Specimens: 7 day cured, 1 day immersed
1% lime A
3% lime A
3% lime B
1% lime B
Control
(d) Soil: Illinois clay
Specimens: 28 day cured, 1 day immersed
Control
0.25% MgO
0.5% Na_2SO_4
0.5% Na_4SiO_4
0.5% $MgSO_4$
(e) Soil: Illinois clay
Specimens 28 day cured, 1 day immersed
0.25% NaOH
Control
0.5%
0.25% Na_2CO_3
$CaSO_4$
0.5%
0.25% $CaCl_2$
1.5%
(f) Soil: Illinois clay
Specimens: 28 day cured, 1 day immersed
3% lime B
Control
1% lime A
3% lime A
1% lime B
Unconfined compressive strength, psi
800
600
400
200
0
4
8
12
Cement content, %

Figure 16. Unconfined compressive strengths of Texas clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents

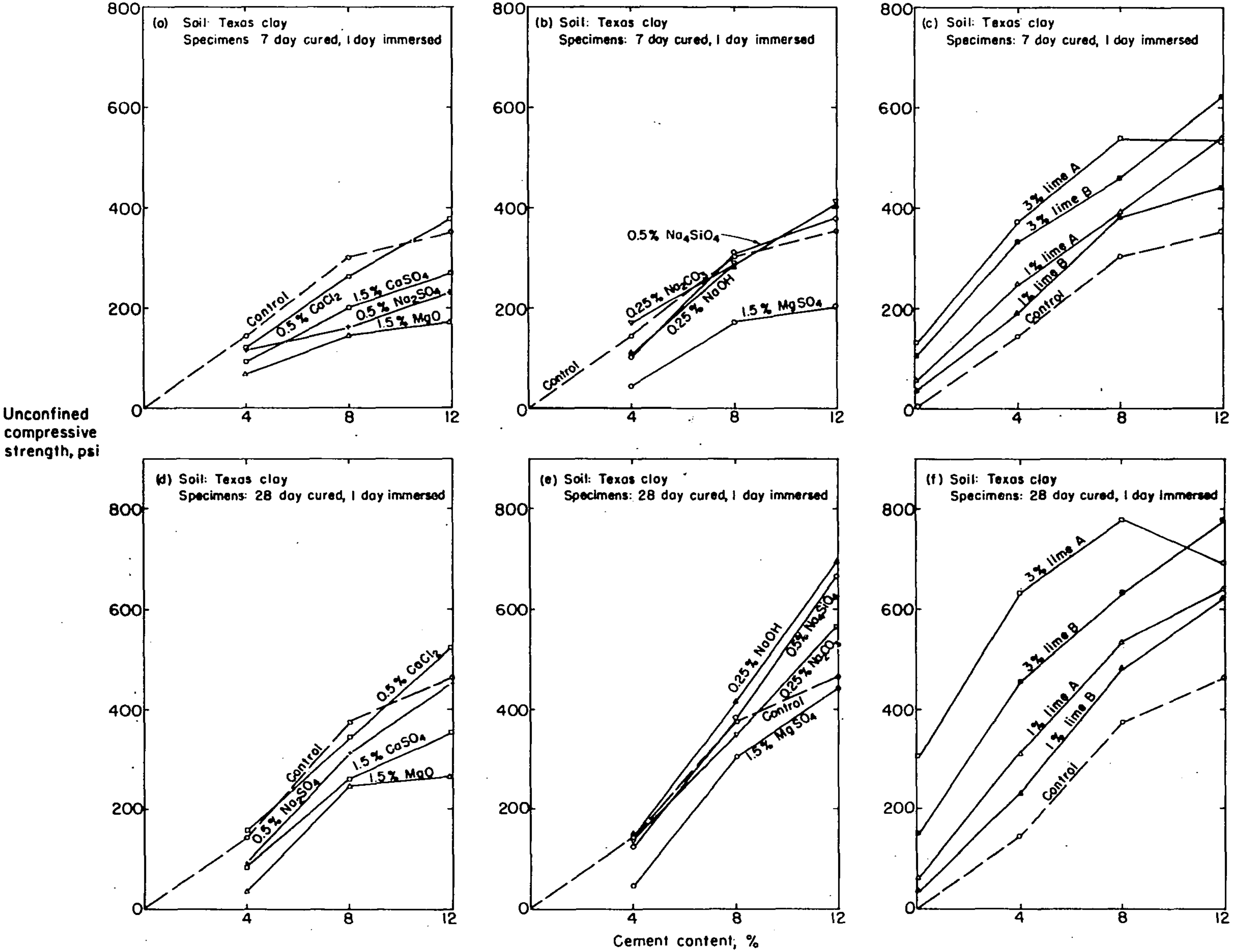

(a) Soil: Texas clay
Specimens 7 day cured, 1 day immersed
Control
0.5% CaCl2
1.5% CaSO4
0.5% Na2SO4
1.5% MgO
(b) Soil: Texas clay
Specimens: 7 day cured, 1 day immersed
0.5% Na4SiO4
0.25% Na2CO3
0.25% NaOH
1.5% MgSO4
Control
(c) Soil: Texas clay
Specimens: 7 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
(d) Soil: Texas clay
Specimens: 28 day cured, 1 day immersed
0.5% CaCl2
Control
0.5% Na2SO4
1.5% CaSO4
1.5% MgO
(e) Soil: Texas clay
Specimens: 28 day cured, 1 day immersed
0.25% NaOH
0.5% Na4SiO4
0.25% Na2CO3
Control
1.5% MgSO4
(f) Soil: Texas clay
Specimens: 28 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
Unconfined compressive strength, psi
Cement content, %
0
200
400
600
800
4
8
12

Figure 17. Unconfined compressive strengths of Michigan clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents

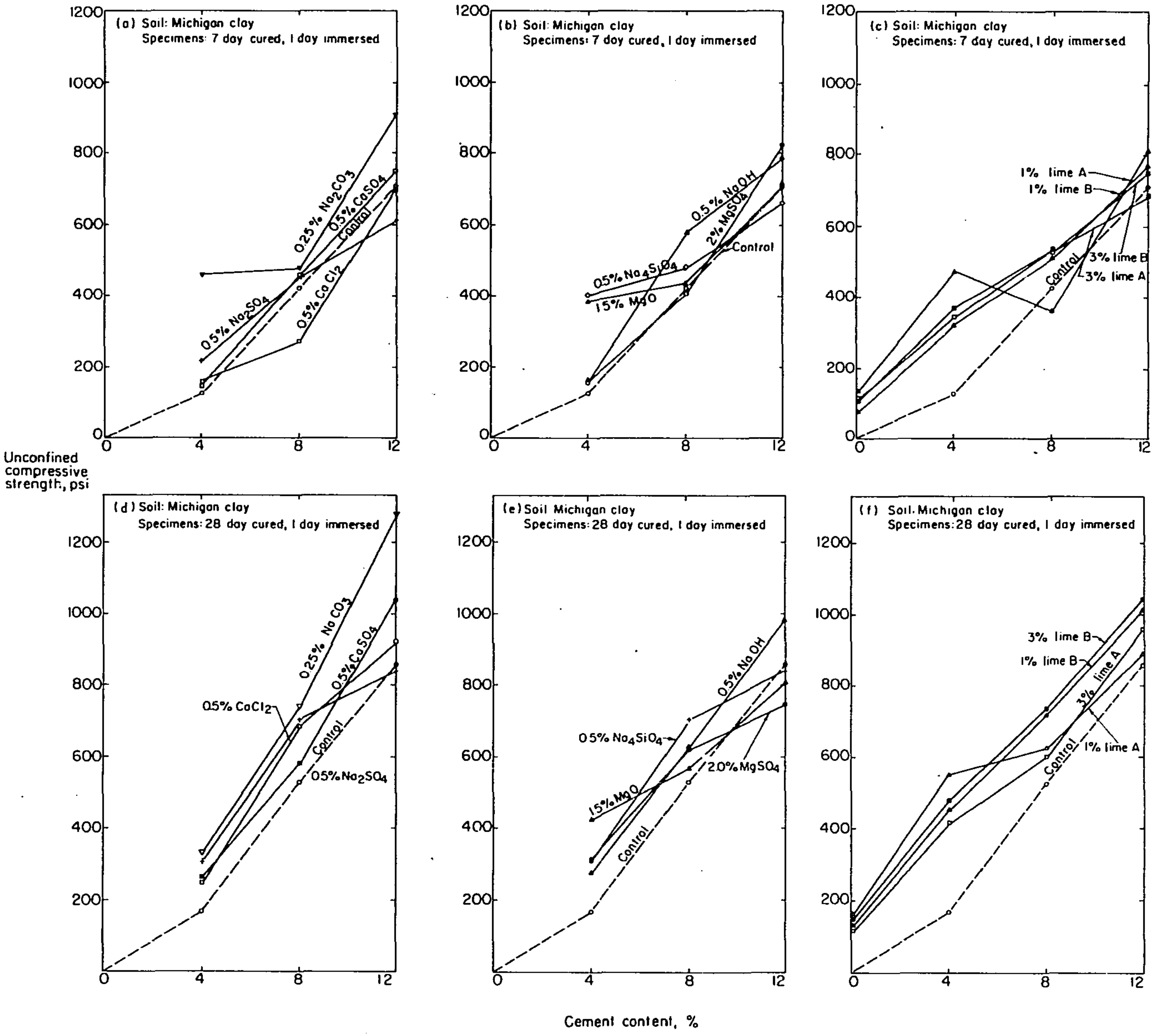

(a) Soil: Michigan clay
Specimens: 7 day cured, 1 day immersed
0.25% Na2CO3
0.5% CaSO4
Control
0.5% CaCl2
0.5% Na2SO4
(b) Soil: Michigan clay
Specimens: 7 day cured, 1 day immersed
0.5% NaOH
2% MgSO4
Control
0.5% Na4SiO4
1.5% MgO
(c) Soil: Michigan clay
Specimens: 7 day cured, 1 day immersed
1% lime A
1% lime B
3% lime B
3% lime A
Control
Unconfined compressive strength, psi
(d) Soil: Michigan clay
Specimens: 28 day cured, 1 day immersed
0.25% Na CO3
0.5% CaSO4
0.5% CaCl2
Control
0.5% Na2SO4
(e) Soil Michigan clay
Specimens: 28 day cured, 1 day immersed
0.5% NaOH
0.5% Na4SiO4
1.5% MgO
2.0% MgSO4
Control
(f) Soil: Michigan clay
Specimens: 28 day cured, 1 day immersed
3% lime B
1% lime B
3% lime A
1% lime A
Control
0
200
400
600
800
1000
1200
0
4
8
12
Cement content, %

Figure 18. Unconfined compressive strengths of North Carolina clay stabilized with optimum amount of chemical or hydrated lime at varying cement contents

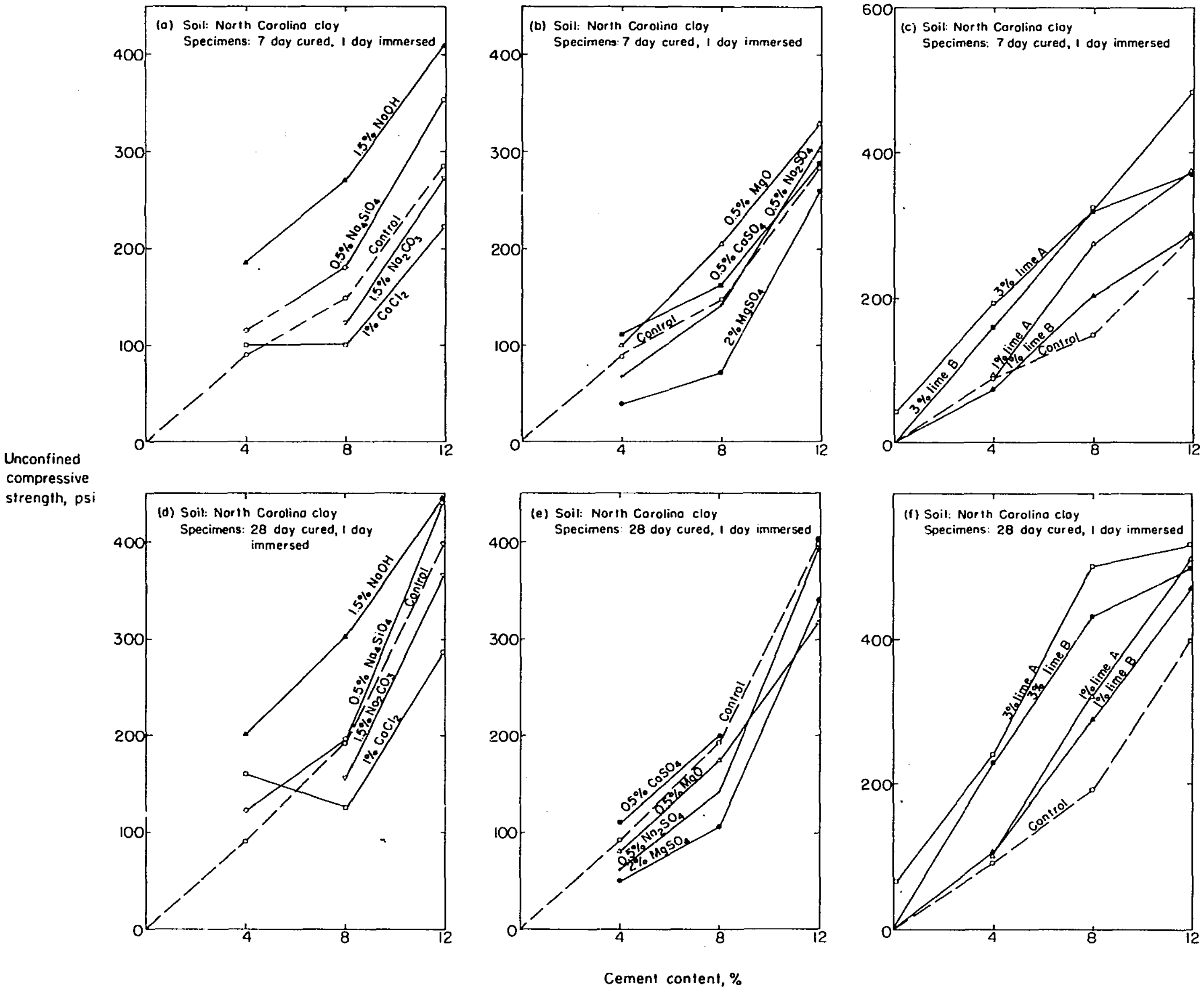

Unconfined compressive strength, psi
Cement content, %
(a) Soil: North Carolina clay
Specimens: 7 day cured, 1 day immersed
1.5% NaOH
0.5% Na4SiO4
Control
1.5% Na2CO3
1% CaCl2
(b) Soil: North Carolina clay
Specimens: 7 day cured, 1 day immersed
0.5% MgO
0.5% CaSO4
0.5% Na2SO4
Control
2% MgSO4
(c) Soil: North Carolina clay
Specimens: 7 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
(d) Soil: North Carolina clay
Specimens: 28 day cured, 1 day immersed
1.5% NaOH
0.5% Na4SiO4
Control
1.5% Na2CO3
1% CaCl2
(e) Soil: North Carolina clay
Specimens: 28 day cured, 1 day immersed
0.5% CaSO4
0.5% MgO
Control
0.5% Na2SO4
2% MgSO4
(f) Soil: North Carolina clay
Specimens: 28 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
0
100
200
300
400
600
4
8
12

Figure 19. Unconfined compressive strengths of Washington sand stabilized with optimum amount of chemical or hydrated lime at varying cement contents

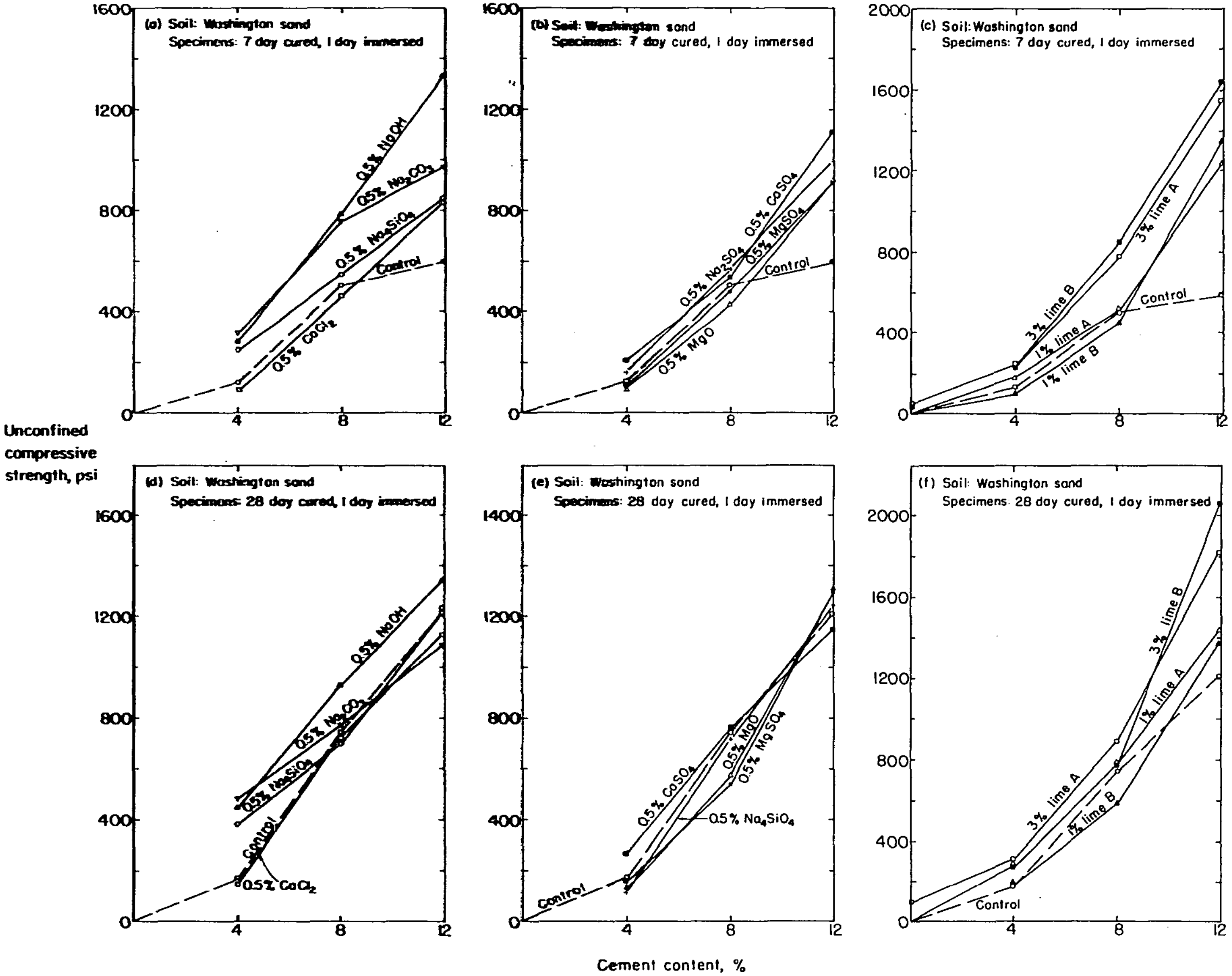

(a) Soil: Washington sand
Specimens: 7 day cured, 1 day immersed
0.5% NaOH
0.5% Na_2CO_3
0.5% Na_4SiO_4
Control
0.5% $CaCl_2$
(b) Soil: Washington sand
Specimens: 7 day cured, 1 day immersed
0.5% $CaSO_4$
0.5% $MgSO_4$
0.5% Na_2SO_4
Control
0.5% MgO
(c) Soil: Washington sand
Specimens: 7 day cured, 1 day immersed
3% lime A
3% lime B
1% lime A
1% lime B
Control
(d) Soil: Washington sand
Specimens: 28 day cured, 1 day immersed
0.5% NaOH
0.5% Na_2CO_3
0.5% Na_4SiO_4
Control
0.5% $CaCl_2$
(e) Soil: Washington sand
Specimens: 28 day cured, 1 day immersed
0.5% $CaSO_4$
0.5% MgO
0.5% $MgSO_4$
0.5% Na_4SiO_4
Control
(f) Soil: Washington sand
Specimens: 28 day cured, 1 day immersed
3% lime B
1% lime A
3% lime A
1% lime B
Control
Unconfined compressive strength, psi
Cement content, %
0
400
800
1200
1400
1600
2000
4
8
12

Figure 20. Unconfined compressive strengths of Iowa silt stabilized with optimum amount of chemical at varying hydrated lime contents

Figure 21. Unconfined compressive strengths of Iowa clay stabilized with optimum amount of chemical at varying hydrated lime contents

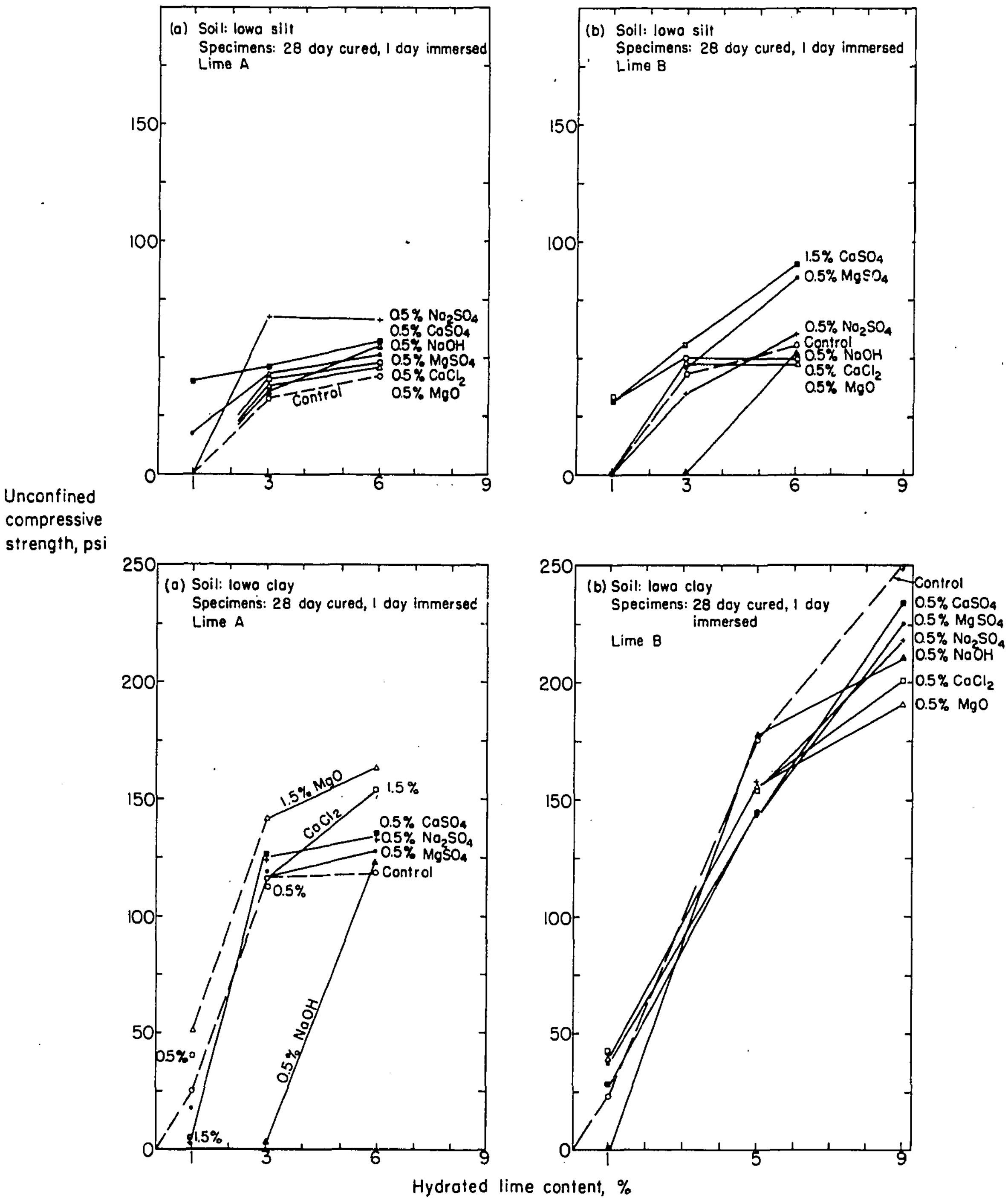

(a) Soil: Iowa silt
Specimens: 28 day cured, 1 day immersed
Lime A
0.5% Na_2SO_4
0.5% $CaSO_4$
0.5% NaOH
0.5% $MgSO_4$
0.5% $CaCl_2$
0.5% MgO
Control
(b) Soil: Iowa silt
Specimens: 28 day cured, 1 day immersed
Lime B
1.5% $CaSO_4$
0.5% $MgSO_4$
0.5% Na_2SO_4
Control
0.5% NaOH
0.5% $CaCl_2$
0.5% MgO
Unconfined compressive strength, psi
(a) Soil: Iowa clay
Specimens: 28 day cured, 1 day immersed
Lime A
1.5% MgO
$CaCl_2$
1.5%
0.5% $CaSO_4$
0.5% Na_2SO_4
0.5% $MgSO_4$
Control
0.5%
0.5% NaOH
0.5%
1.5%
(b) Soil: Iowa clay
Specimens: 28 day cured, 1 day immersed
Lime B
Control
0.5% $CaSO_4$
0.5% $MgSO_4$
0.5% Na_2SO_4
0.5% NaOH
0.5% $CaCl_2$
0.5% MgO
0
50
100
150
200
250
1
3
5
6
9
Hydrated lime content, %

Figure 22. Unconfined compressive strengths of Wisconsin sand stabilized with optimum amount of chemical at varying hydrated lime contents

Figure 23. Unconfined compressive strengths of Illinois clay stabilized with optimum amount of chemical at varying hydrated lime contents

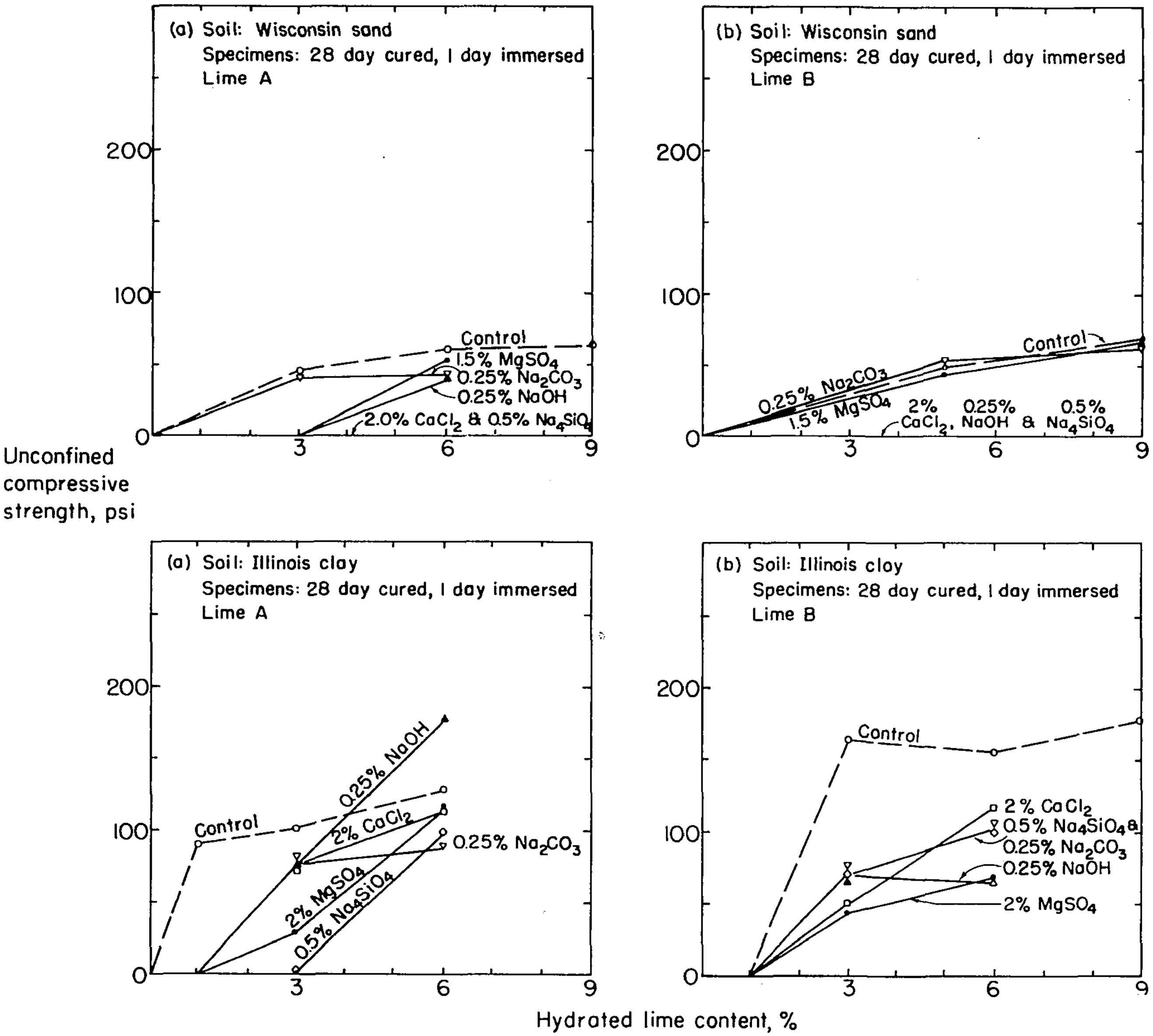

(a) Soil: Wisconsin sand
Specimens: 28 day cured, 1 day immersed
Lime A
Control
1.5% $MgSO_4$
0.25% Na_2CO_3
0.25% NaOH
2.0% $CaCl_2$ & 0.5% Na_4SiO_4
(b) Soil: Wisconsin sand
Specimens: 28 day cured, 1 day immersed
Lime B
Control
0.25% Na_2CO_3
1.5% $MgSO_4$
2% $CaCl_2$, 0.25% NaOH & 0.5% Na_4SiO_4
Unconfined compressive strength, psi
(a) Soil: Illinois clay
Specimens: 28 day cured, 1 day immersed
Lime A
Control
0.25% NaOH
2% $CaCl_2$
2% $MgSO_4$
0.5% Na_4SiO_4
0.25% Na_2CO_3
(b) Soil: Illinois clay
Specimens: 28 day cured, 1 day immersed
Lime B
Control
2% $CaCl_2$
0.5% Na_4SiO_4 &
0.25% Na_2CO_3
0.25% NaOH
2% $MgSO_4$
0
100
200
3
6
9
Hydrated lime content, %

Figure 24. Unconfined compressive strengths of Texas clay stabilized with optimum amount of chemical at varying hydrated lime contents

Figure 25. Unconfined compressive strengths of Michigan clay stabilized with optimum amount of chemical at varying hydrated lime contents

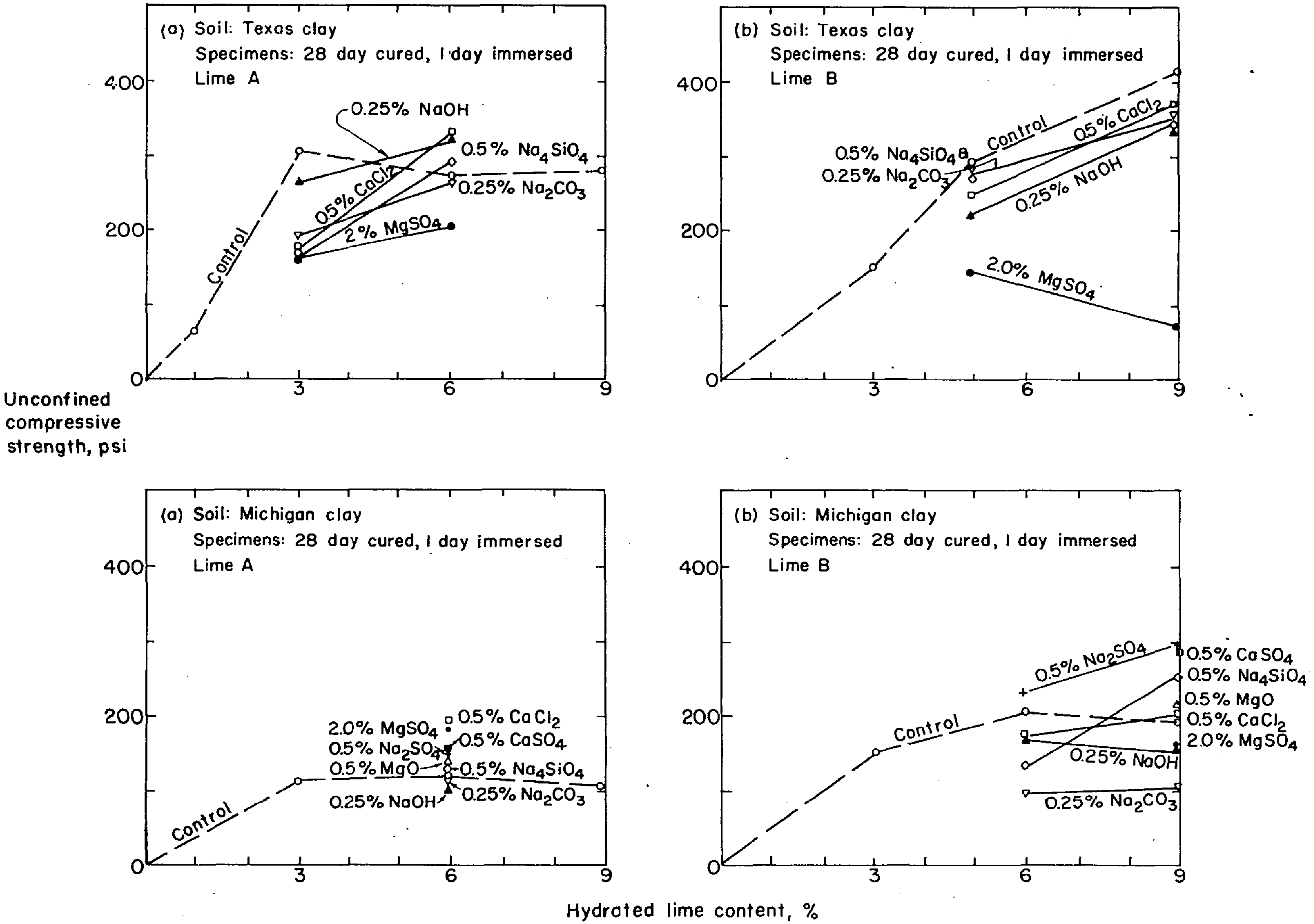
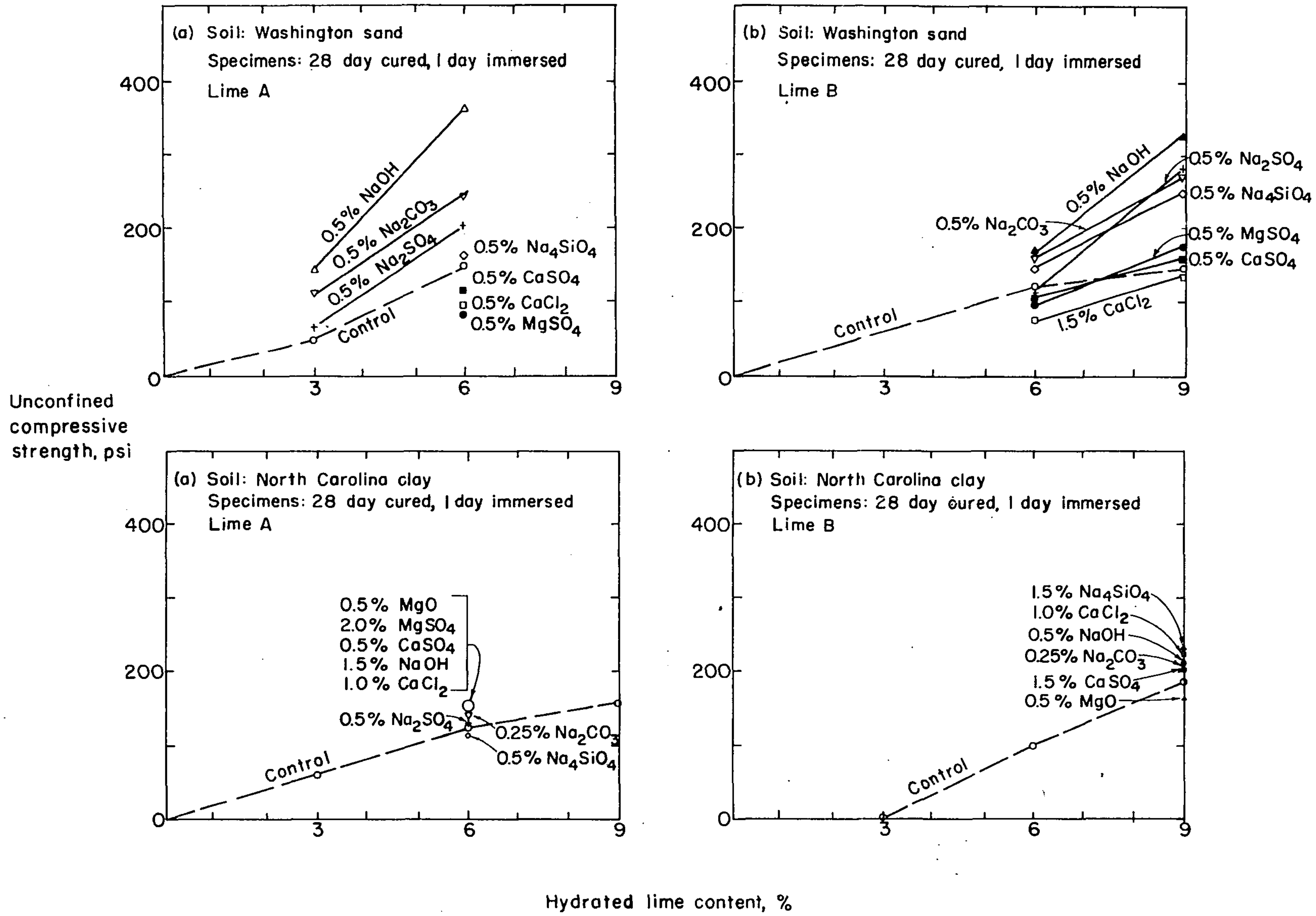

(a) Soil: Texas clay
Specimens: 28 day cured, 1 day immersed
Lime A
0.25% NaOH
0.5% Na_4SiO_4
0.5% $CaCl_2$
0.25% Na_2CO_3
2% $MgSO_4$
Control
(b) Soil: Texas clay
Specimens: 28 day cured, 1 day immersed
Lime B
Control
0.5% $CaCl_2$
0.5% Na_4SiO_4
0.25% Na_2CO_3
0.25% NaOH
2.0% $MgSO_4$
(a) Soil: Michigan clay
Specimens: 28 day cured, 1 day immersed
Lime A
2.0% $MgSO_4$
0.5% $CaCl_2$
0.5% Na_2SO_4
0.5% $CaSO_4$
0.5% MgO
0.5% Na_4SiO_4
0.25% NaOH
0.25% Na_2CO_3
Control
(b) Soil: Michigan clay
Specimens: 28 day cured, 1 day immersed
Lime B
0.5% Na_2SO_4
0.5% $CaSO_4$
0.5% Na_4SiO_4
0.5% MgO
0.5% $CaCl_2$
2.0% $MgSO_4$
0.25% NaOH
0.25% Na_2CO_3
Control
Unconfined compressive strength, psi
0
200
400
0
3
6
9
Hydrated lime content, %

Figure 26. Unconfined compressive strengths of Washington sand stabilized with optimum amount of chemical at varying hydrated lime contents

Figure 27. Unconfined compressive strengths of North Carolina clay stabilized with optimum amount of chemical at varying hydrated lime contents


(a) Soil: Washington sand
Specimens: 28 day cured, 1 day immersed
Lime A
0.5% NaOH
0.5% Na_2CO_3
0.5% Na_2SO_4
0.5% Na_4SiO_4
0.5% $CaSO_4$
0.5% $CaCl_2$
0.5% $MgSO_4$
Control
(b) Soil: Washington sand
Specimens: 28 day cured, 1 day immersed
Lime B
0.5% NaOH
0.5% Na_2CO_3
0.5% Na_2SO_4
0.5% Na_4SiO_4
0.5% $MgSO_4$
0.5% $CaSO_4$
1.5% $CaCl_2$
Control
(a) Soil: North Carolina clay
Specimens: 28 day cured, 1 day immersed
Lime A
0.5% MgO
2.0% $MgSO_4$
0.5% $CaSO_4$
1.5% NaOH
1.0% $CaCl_2$
0.5% Na_2SO_4
0.25% Na_2CO_3
0.5% Na_4SiO_4
Control
(b) Soil: North Carolina clay
Specimens: 28 day cured, 1 day immersed
Lime B
1.5% Na_4SiO_4
1.0% $CaCl_2$
0.5% NaOH
0.25% Na_2CO_3
1.5% $CaSO_4$
0.5% MgO
Control
Unconfined compressive strength, psi
0
200
400
0
3
6
9
Hydrated lime content, %

APPENDIX C

Table 20. Durability indices for Iowa silt stabilized with 12 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	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 21. Durability indices for Iowa clay stabilized with 8 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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

Table 22. Durability indices for Iowa clay stabilized with 12 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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 23. Durability indices for Wisconsin sand stabilized with 12 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	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

Table 24. Durability indices for Wisconsin sand stabilized with 16 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	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 25. Durability indices for Illinois clay stabilized with 4 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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

Table 26. Durability indices for Illinois clay stabilized with 8 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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

Table 27. Durability indices for Illinois clay stabilized with 12 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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	1092	85	918	1122	82
Lime B	3.00	1000	1058	95	860	1070	80

Table 28. Durability indices for Texas clay stabilized with 4 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	113	156	73	139	261	53
Calcium chloride	0.50			----			
Sodium carbonate	0.25			----			
Sodium hydroxide	0.25			----			
Sodium orthosilicate	0.50			----			
Lime A	3.00	399	360	111	363	478	76
Lime B	3.00	471	461	102	415	544	76

Table 29. Durability indices for Texas clay stabilized with 8 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	402	432	93	458	521	88
Calcium chloride	0.50			----			
Sodium carbonate	0.25			----			
Sodium hydroxide	0.25	389	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

Table 30. Durability indices for Texas clay stabilized with 12 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	455	501	92	492	570	86
Calcium chloride	0.50	455	523	86	452	528	86
Sodium carbonate	0.25	471	468	101	508	604	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	85
Lime B	3.00	623	778	80	653	798	82

Table 31. Durability indices for Texas clay stabilized with 3 percent lime A

Additive Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	139	201	69	212	346	61
Calcium chloride	0.50	119	188	63	228	313	73
Sodium orthosilicate	0.50	121	172	70	145	178	84

Table 32. Durability indices for Texas clay stabilized with 6 percent lime A

Additive Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	0	396	0	360	448	75
Calcium chloride	0.50	383	488	78	396	531	75
Sodium orthosilicate	0.50	284	379	75	366	412	89

Table 33. Durability indices for Texas clay stabilized with 9 percent lime B

Additive Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	212	369	58	353	504	70
Calcium chloride	0.50	281	337	84	402	498	81
Sodium orthosilicate	0.50	369	419	88	442	554	80

Table 34. Durability indices for Michigan clay stabilized with 4 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	138	193	72	158	202	78
Calcium chloride	0.50	123	113	109	139	208	67
Calcium sulfate	0.50	---					
Magnesium oxide	1.50	240	254	95	316	370	87
Magnesium sulfate	2.00	---					
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	188	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	267	89	251	389	65

Table 35. Durability indices for Michigan clay stabilized with 8 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	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

Table 36. Durability indices for Michigan clay stabilized with 12 percent cement

Additive Type	Additive Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	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	1020	1140	98
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 37. Durability indices for North Carolina clay stabilized with 4 percent cement

Additive							
Type	Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	0	0	0	61	78	78
Calcium chloride	1.00	55	88	63	78	90	87
Calcium sulfate	0.50	----					
Magnesium oxide	0.50	----					
Sodium carbonate	1.50	----					
Sodium hydroxide	1.50	72	90	80	93	120	78
Sodium orthosilicate	0.50	----					
Sodium sulfate	0.50	----					
Lime A	3.00	----					
Lime B	3.00	----					

Table 38. Durability indices for North Carolina clay stabilized with 8 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	104	133	78	140	184	76
Calcium chloride	1.00	58	85	68	75	93	81
Calcium sulfate	0.50	----					
Magnesium oxide	0.50	----					
Sodium carbonate	1.50	----					
Sodium hydroxide	1.50	188	175	103	165	192	86
Sodium orthosilicate	0.50	----					
Sodium sulfate	0.50	----					
Lime A	3.00	366	353	104	337	383	88
Lime B	3.00	261	277	94	205	251	82

Table 39. Durability indices for North Carolina clay stabilized with 12 percent cement

Additive Type	Additive Content, %	P_f psi	P_{c_f} psi	R_f %	P_i psi	P_{c_i} psi	R_i %
----	----	337	320	105	343	294	117
Calcium chloride	1.00	243	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	393	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 40. Durability indices for North Carolina clay stabilized with 9 percent lime B

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	96	104	92	148	203	73
Calcium chloride	1.00	90	107	84	104	112	93
Calcium sulfate	1.50	265	311	85	297	342	87

Table 41. Durability indices for Washington sand stabilized with 4 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	102	133	77	136	158	86
Calcium chloride	0.50	97	100	97	128	161	80
Calcium sulfate	0.50	----					
Magnesium oxide	0.50	----					
Magnesium sulfate	0.50	----					
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	264	64
Lime B	3.00	231	248	93	287	320	90

Table 42. Durability indices for Washington sand stabilized with 8 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	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
Magnesium oxide	0.50			----			
Magnesium sulfate	0.50			----			
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

Table 43. Durability indices for Washington sand stabilized with 12 percent cement

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	522	784	67	942	1183	80
Calcium chloride	0.50	697	840	83	886	1043	85
Calcium sulfate	0.50	963	1048	92	1039	1142	91
Magnesium oxide	0.50	717	850	84	783	920	85
Magnesium sulfate	0.50	573	628	91	1282	1296	99
Sodium carbonate	0.50	723	872	83	824	984	84
Sodium hydroxide	0.50	1109	1224	91	1271	1352	94
Sodium orthosilicate	0.50	719	958	75	1008	1008	100
Sodium sulfate	0.50	902	894	101	912	1153	79
Lime A	3.00	1382	1595	87	1545	1770	87
Lime B	3.00	1472	1667	89	1709	1978	86

Table 44. Durability indices for Washington sand stabilized with 6 percent lime A

Additive		P_f	P_{c_f}	R_f	P_i	P_{c_i}	R_i
Type	Content, %	psi	psi	%	psi	psi	%
----	----	43	97	44	107	172	62
Calcium chloride	0.50	42	97	43	83	80	104
Sodium hydroxide	0.50	258	292	88	312	341	91

APPENDIX D

Figure 28. Mohr diagrams for tests performed on Iowa silt mixes

Figure 29. Mohr diagrams for tests performed on Iowa clay mixes

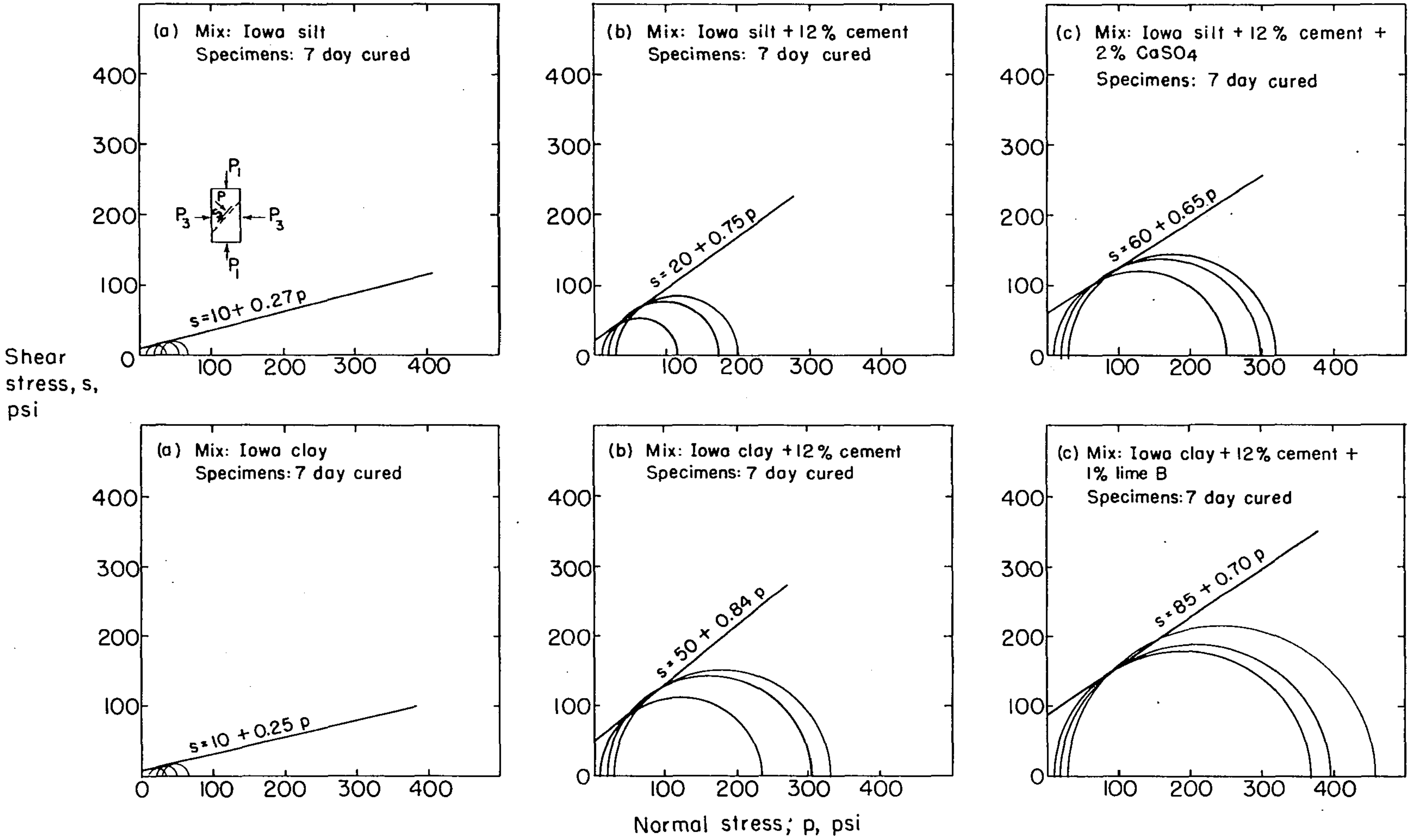

(a) Mix: Iowa silt
Specimens: 7 day cured
s=10+ 0.27 p
P_1
P_3
p
s
(b) Mix: Iowa silt + 12 % cement
Specimens: 7 day cured
s=20 + 0.75 p
(c) Mix: Iowa silt + 12 % cement + 2 % CaSO4
Specimens: 7 day cured
s=60 + 0.65 p
(a) Mix: Iowa clay
Specimens: 7 day cured
s=10 + 0.25 p
(b) Mix: Iowa clay + 12 % cement
Specimens: 7 day cured
s=50 + 0.84 p
(c) Mix: Iowa clay + 12 % cement + 1% lime B
Specimens: 7 day cured
s=85 + 0.70 p
Shear stress, s, psi
Normal stress; p, psi
0
100
200
300
400

Figure 30. Mohr diagrams for tests performed on Wisconsin sand mixes

Figure 31. Mohr diagrams for tests performed on Illinois clay mixes

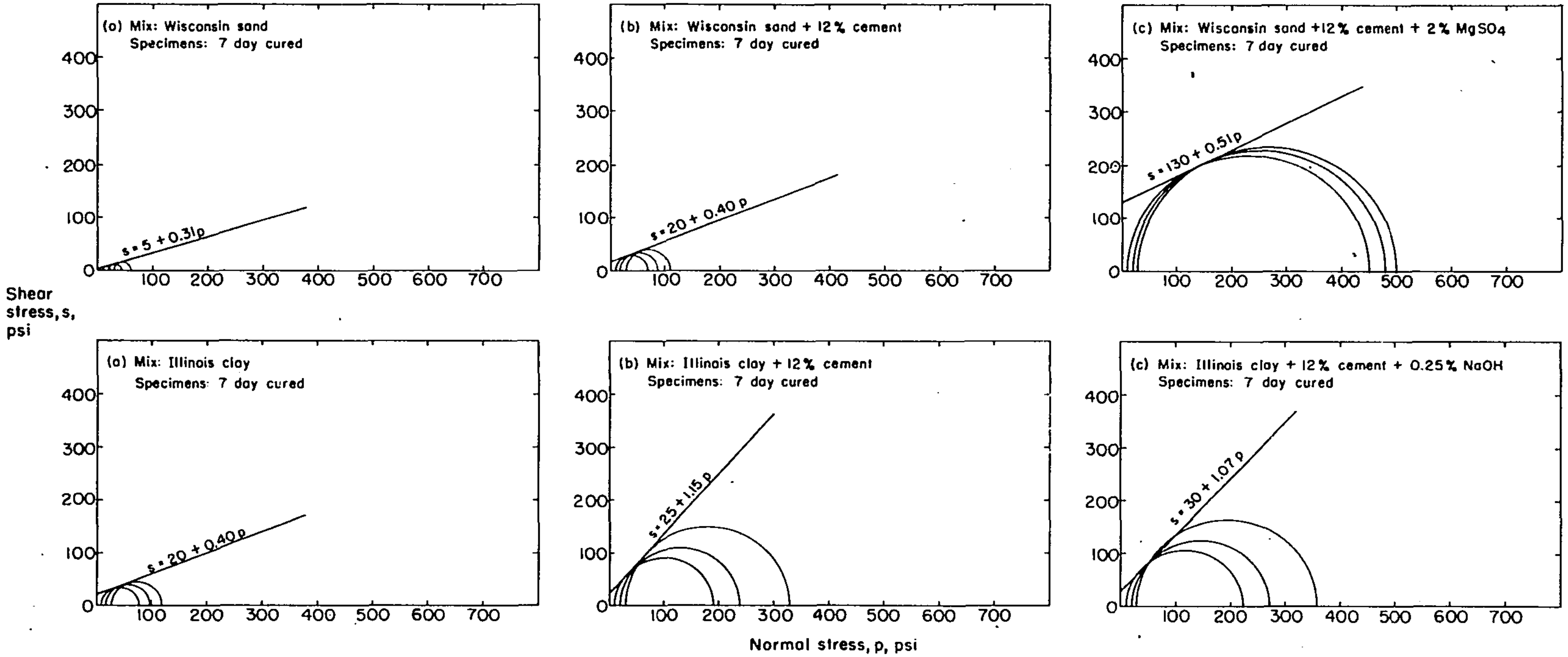

(a) Mix: Wisconsin sand
Specimens: 7 day cured
s = 5 + 0.31p
(b) Mix: Wisconsin sand + 12% cement
Specimens: 7 day cured
s = 20 + 0.40 p
(c) Mix: Wisconsin sand +12% cement + 2% MgSO4
Specimens: 7 day cured
s = 130 + 0.51p
(a) Mix: Illinois clay
Specimens: 7 day cured
s = 20 + 0.40p
(b) Mix: Illinois clay + 12% cement
Specimens: 7 day cured
s = 25 + 1.15 p
(c) Mix: Illinois clay + 12% cement + 0.25% NaOH
Specimens: 7 day cured
s = 30 + 1.07 p
Shear stress, s, psi
Normal stress, p, psi
0
100
200
300
400
500
600
700

Figure 32. Mohr diagrams for tests performed on Texas clay mixes

Figure 33. Mohr diagrams for tests performed on Michigan clay mixes

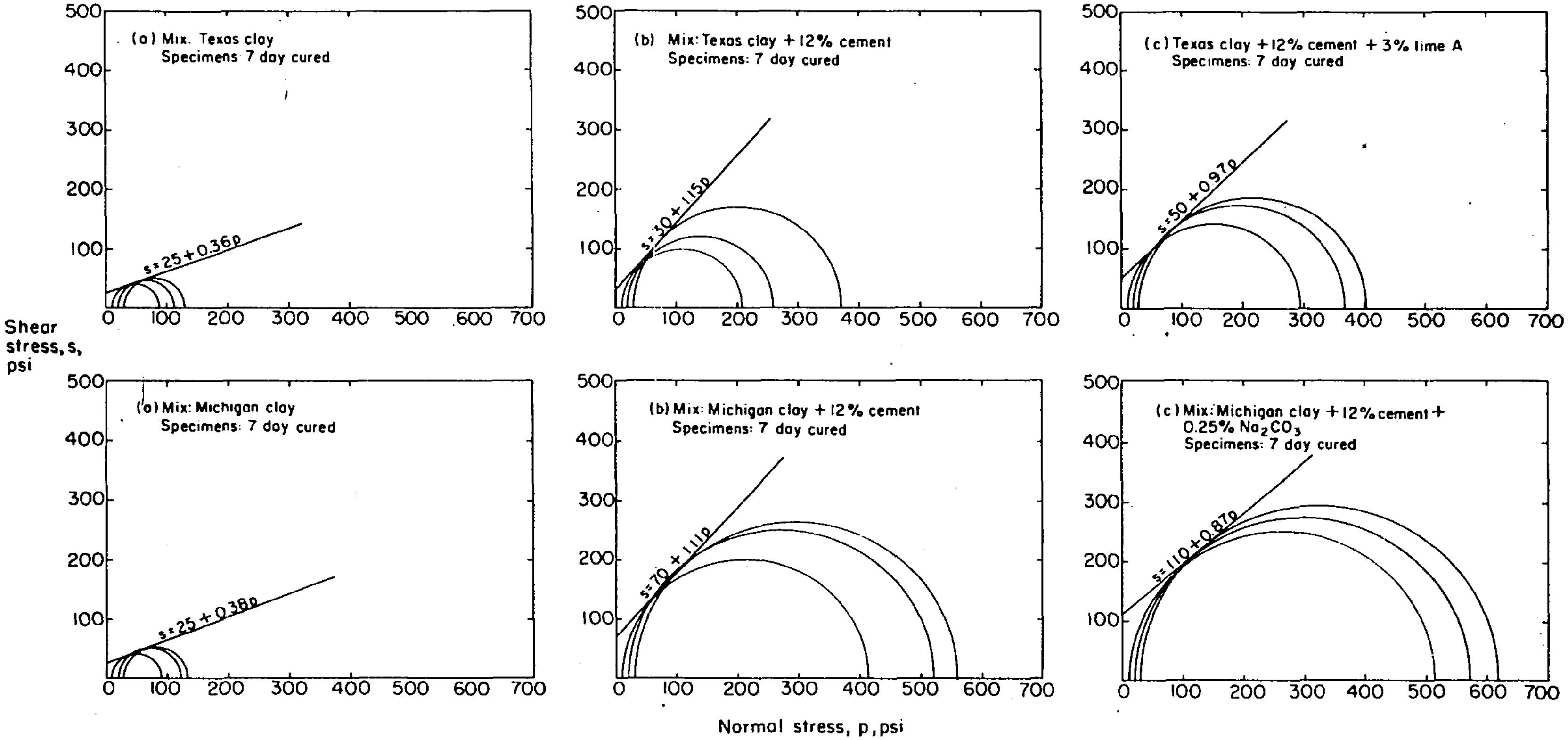

Shear stress, s, psi
Normal stress, p, psi
(a) Mix. Texas clay
Specimens 7 day cured
s = 25 + 0.36p
(b) Mix: Texas clay + 12% cement
Specimens: 7 day cured
s = 30 + 1.15p
(c) Texas clay + 12% cement + 3% lime A
Specimens: 7 day cured
s = 50 + 0.97p
(a) Mix: Michigan clay
Specimens: 7 day cured
s = 25 + 0.38p
(b) Mix: Michigan clay + 12% cement
Specimens: 7 day cured
s = 70 + 1.11p
(c) Mix: Michigan clay + 12% cement + 0.25% Na2CO3
Specimens: 7 day cured
s = 110 + 0.87p
0
100
200
300
400
500
600
700

Figure 34. Mohr diagrams for tests performed on North Carolina clay mixes

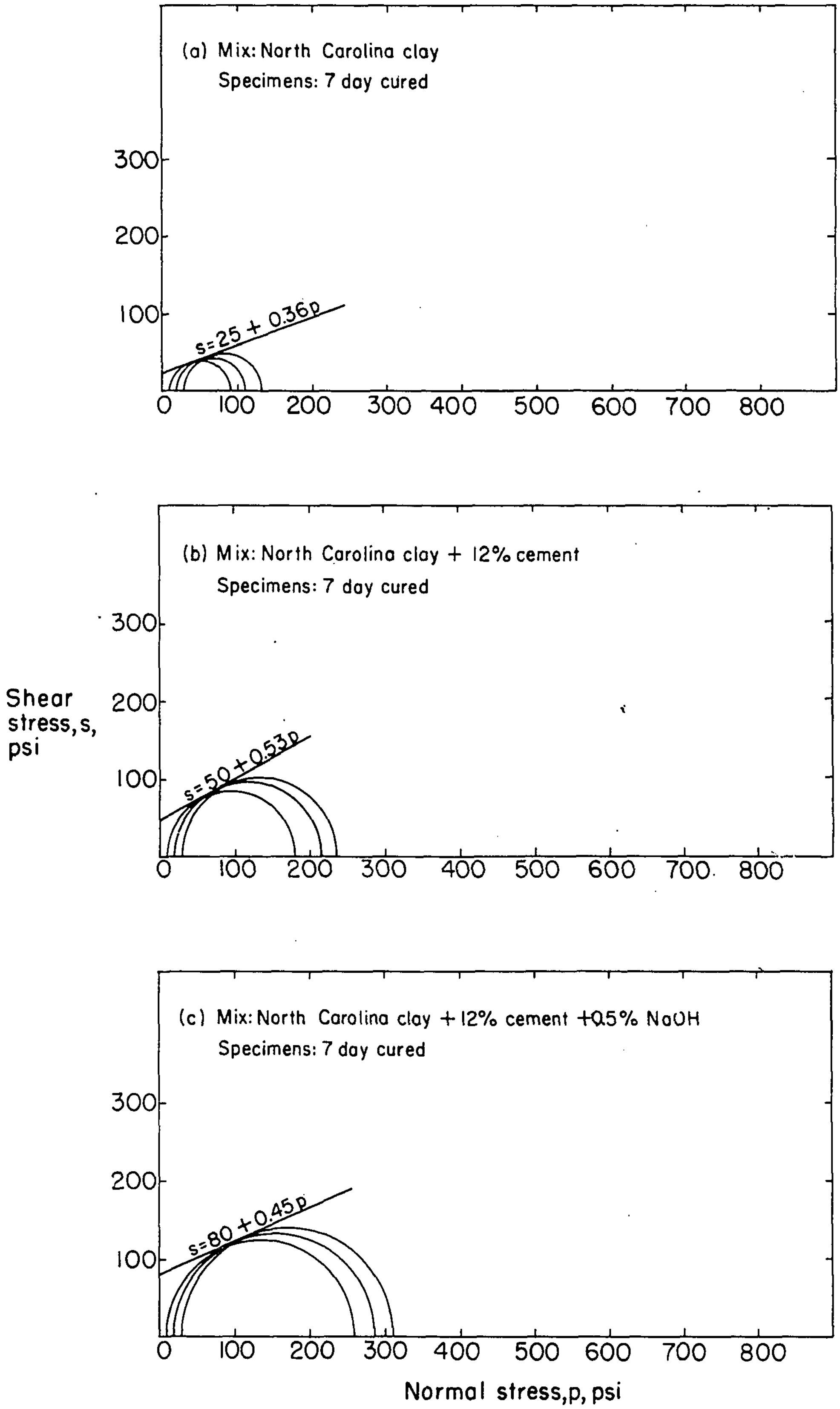
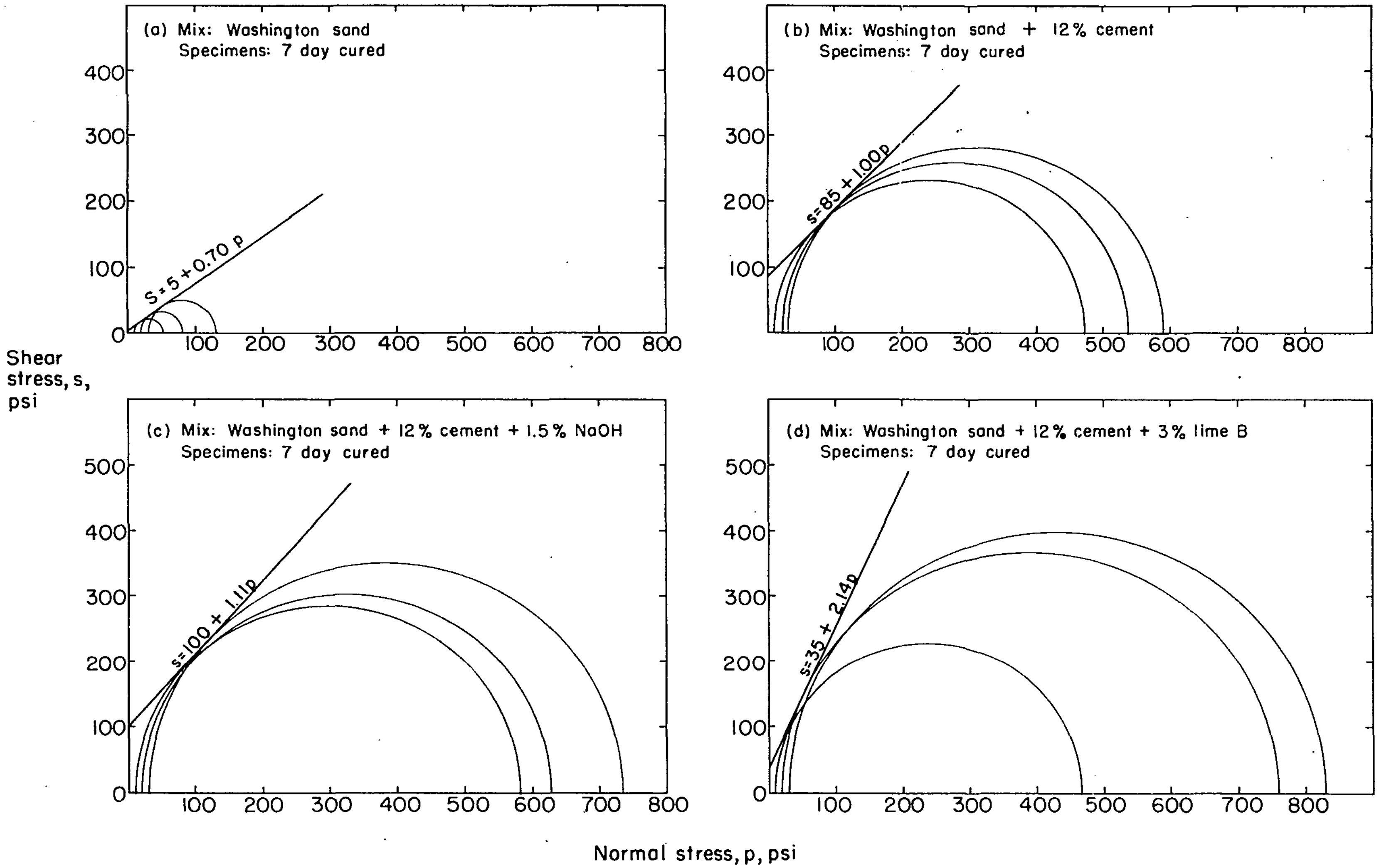

(a) Mix: North Carolina clay
Specimens: 7 day cured
s=25 + 0.36p
(b) Mix: North Carolina clay + 12% cement
Specimens: 7 day cured
s=50+0.53p
(c) Mix: North Carolina clay + 12% cement +0.5% NaOH
Specimens: 7 day cured
s=80 +0.45p
Shear stress, s, psi
100
200
300
0
100
200
300
400
500
600
700
800
Normal stress, p, psi

Figure 35. Mohr diagrams for tests performed on Washington sand mixes


(a) Mix: Washington sand
Specimens: 7 day cured
S=5+0.70 p
(b) Mix: Washington sand + 12% cement
Specimens: 7 day cured
s=85+1.00p
(c) Mix: Washington sand + 12% cement + 1.5% NaOH
Specimens: 7 day cured
s=100+1.11p
(d) Mix: Washington sand + 12% cement + 3% lime B
Specimens: 7 day cured
s=35+2.14p
0
100
200
300
400
500
600
700
800
Shear stress, s, psi
Normal stress, p, psi