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Cement-Soil Stabilization

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2101 Constitution Avenue — Washington 25, D. C.

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Cement-Soil Stabilization

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Use of Flyash with Portland Cement for Stabilization of Soils

D. T. DAVIDSON, Professor of Civil Engineering,
R. K. KATTI, Research Graduate Assistant,
Iowa Engineering Experiment Station, Iowa State College, Ames, and
D. E. WELCH, First Lieutenant, Corps of Engineers, U.S. Army

This paper presents the results of a laboratory investigation of soil stabilization with portland cement and flyash. The flyash was used as a partial replacement for portland cement or as an additive to mixtures already meeting the requirements of soil-cement. Seven-day, 28-day and 120-day unconfined compressive strength results were used as criteria for evaluating the soil-cement-flyash mixtures.

As an additive to soil-cement mixture, flyash increased the 120-day unconfined compressive strength of sand-cement mixtures. Similar effect was observed with friable loess-cement-flyash mixes containing amounts of cement equal to or higher than the soil-cement requirement.

In the case of plastic loess and alluvial clay little strength gain was realized with flyash as an additive. However, 120-day strength results show that shrinkage cracking in clay-cement mixes is appreciable reduced by the addition of flyash. Addition of flyash to soil-cement mixtures of friable loess did not improve the freeze and thaw resistance of the mixtures.

A part of the cement can be replaced with flyash in soil-cement mixtures of friable loess-cement without loss of strength. The replacement of cement with flyash resulted in reduction in strength of the clay-cement flyash mixes except at 120-days, where strength remains higher due to reduced shrinkage cracking.

An attempt is made to interpret test results on the basis of pozzolanic activity with the lime liberated during cement hydration.

● ONE OF THE MOST common methods of soil stabilization in use today is with portland cement. In this method pulverized soil is mixed with a predetermined amount of cement (water is added during mixing if the moisture content is less than required for compaction) and the mixture is shaped, compacted and moist-cured. The resulting product is "soil-cement," this term implying that the mixture is designed to meet specifications of strength and durability established by the Portland Cement Association (1). Soil-cement, properly designed and constructed, has a world-wide record of satisfactory field performance as a base course material for roads and airfields

Despite the commendable performance record of soil-cement, there is still room for improvement in its properties and in its economy of use, particularly with silty and clayey soils. Property improvements to be desired include decreased shrinkage and permeability and increased resistance to freezing and thawing. A reduction of of the cement requirement without sacrificing needed strength and durability is desirable from the standpoint of both cost and the limited availability of cement in many areas during the last few years. The possibility of using flyash to improve the properties and/or economy of soil-cement was suggested by the extensive use of flyash in concrete for these purposes.

Flyash in Concrete

Flyash is the most commonly used commercial pozzolan[1]. It is collected in large

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[1] A pozzolan is defined in ASTM Standard Definition of Terms Relating to Hydraulic

TABLE 1

BRIEF DESCRIPTION OF SOILS USED

	Friable loess (Lab. No. 20-2 (V)	Plastic loess (Lab. No. 44A-1)	Alluvial clay (Lab. No. A1-1)	Dune sand (Lab. No. S-6-2)
Location	Harrison County, SW Iowa	Page County, SW Iowa	Woodbury County, W Iowa	Benton County, E Iowa
Geological description	Wisconsin age friable loess, oxidized Thickness over 100 ft.	Wisconsin age plastic loess, oxidized Total thickness 15-20 ft.	Recent backswamp clay from Missouri River. Thickness undetermined.	Wisconsin age eolian sand, fine-grained oxidized. Thickness over 20 ft.
Soil series	Hamburg	Marshall	Luton	Carrington
Horizon	C	C	A-C	C
Sampling depth	39 - 40 ft	4 - 5 ft	0 - 3 ft	1½ - 16½ ft

quantities from the smoke in power plants burning powdered coal. Being a waste product flyash is cheap, which explains the widespread interest in utilizing it in concrete, especially as a partial replacement for the more expensive portland cement. Flyash contributes to strength in concrete by reacting with the lime and alkalies liberated by the hydrating cement to produce a gel, perhaps similar to the gel formed by the hydration of portland cement. Since the pozzolanic cementation develops more slowly than cementation from portland cement hydration, 28 days or longer may be required to compensate for the initial strength loss in concrete due to replacing part of the cement with flyash; the strength eventually may exceed that of concrete without flyash (3, 4). According to published reports (3, 4, 5, 6) the following benefits to properties of concrete, particularly mass concrete, may be obtained when flyash is used to replace a portion of either the cement or sand: improved durability, workability and resistance to sulfate attack; decreased permeability, shrinkage, bleeding, evolution of heat and segregation of aggregates; reduced expansion from the reaction between alkalies of the cement and certain types of aggregates. Some of these benefits are desirable for soil-cement.

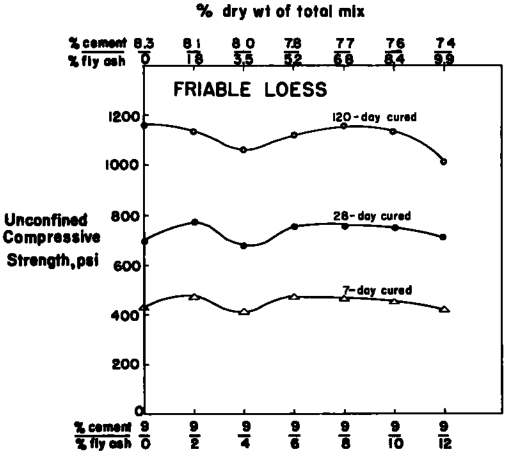

Figure 1. Effect of flyash additives to friable loess-cement on immersed unconfined compressive strength, when the portland cement content is near the minmum requirement for soil-cement.

Purpose of Investigation

Although the use of flyash in portland cement concrete has been extensively investigated, there is little published material available on its use in soil-cement other than the brief report by Baker (7) on the limited studies made by the West Virginia State Road Commission. The purpose of the present laboratory investigation was to explore the possibility of benefiting portland cement stabilization of sandy, silty and clayey soils by using flyash either as a partial replacement for portland cement or as an additive to the soil-cement mixture. Unconfined compressive strength and resistance to freezing and thawing were used as the principal criteria of improvement.

MATERIALS USED

Soils

A description and the properties of the

Cement (2) as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties."

TABLE 2

PROPERTIES OF SOILS USED

	Friable loess	Plastic loess	Alluvial clay	Dune sand
Textural composition, %[a]				
Gravel (>2.0 mm)	0	0	0	0
Sand (2.0 - 0.074 mm)	0.4	0.2	1.5	95.8
Silt (74 - 5μ)	82.6	58.0	24.2	1.2
Clay (<5μ)	17.0	41.8	74.3	3.0
Colloids (<1μ)	12.3	31.0	55.1	2.9
Predominant clay mineral[b]	Montmorillonite and illite	Montmorillonite and illite	Montmorillonite and illite	Montmorillonite and illite
Probable predominant exchangeable cation	Calcium	Calcium	Calcium	Calcium
Specific gravity, 25 C/4 C	2.68	2.72	2.65	2.65
Chemical properties				
Cat. ex. cap., m.e./100 gm[c]	13.4	28.2	39.4	1.5
Carbonates,[d] %	10.2	0.8	2.3	1.4
pH	7.8	6.2	7.3	7.4
Organic matter, %	0.2	0.5	1.7	0.17
Physical properties				
Liquid limit, %	32.9	53.1	71.0	91.0
Plastic limit, %	21.1	25.7	24.5	-
Plasticity index	11.8	27.4	46.5	N.P.
Shrinkage limit, %	28.3	19.9	10.2	-
Centrifuge moist. equiv., %	15.2	21.3	38.4	-
Classification				
Textural[e]	Silty clay loam	Silty clay	Clay	Sand
Engineering (AASHO)	A-4(8)	A-7-6(18)	A-7-6(20)	A-3(0)

[a] Dispersed by air-jet with sodium metaphosphate dispersing agent

[b] From differential thermal analysis of fraction passing No. 200 sieve.

[c] Fraction passing No. 40 sieve.

[d] From differential thermal analysis.

[e] Textural classification is based on former Bureau of Public Roads system (8, p. 18) except that sand and silt sizes are separated on No. 200 sieve (0.074 mm).

four soils used in the investigation are given in Tables 1 and 2. The samples are typical of major fine-grained soil types in Iowa. The most detailed studies were with the friable loess; the plastic loess, alluvial clay and dune sand were used to obtain some test results with different textural types.

Portland Cement and Flyash

Fresh samples of portland cement and flyash were used. Chemical composition and physical property data are given in Table 3.

Portland Cement. The cement was marketed as Type I, the type commonly employed in soil-cement construction. It may be classed as medium-alkali cement (equiv. $Na_2O = 0.49\%$); the tricalcium silicate content of the cement was 53 percent. Tricalcium silicate content is related to the amount of lime liberated during cement hydration, the more tricalcium silicate the more lime that should be available to react with a pozzolan. Type I and Type II cements contain more tricalcium silicate than Type IV. According to Davis (9) a larger replacement of cement by a pozzolan may be made when Type I or Type II cement is used than when Type IV is used. Brink and Halstead (6), working with cement-flyash mortars, found evidence that the alkali in the cement accelerates the pozzolanic reaction at earlier ages, whereas at later ages the amount of tricalcium silicate in the cement governs the benefits derived from the addition of flyash to the mortar.

Flyash. The most reliable criteria for judging the quality of flyash for use as a pozzolan in concrete appear to be the loss on ignition[2] of the flyash and the fineness

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[2] Approximately equal to carbon content.

TABLE 3

CHEMICAL COMPOSITION AND PHYSICAL PROPERTIES OF THE PORTLAND CEMENT AND FLYASH

	Portland Cement[a] (Type I)	Flyash[b]
Chemical composition, %		
Silicon dioxide	21.8	41.9
Aluminum oxide	4.9	22.5
Ferric oxide	2.7	25.8
Calcium oxide	64.3	2.7
Magnesium oxide	2.2	1.0
Sulfur trioxide	2.2	0.8
Loss on ignition	1.0	3.6
Sodium oxide	0.21	0.3
Potassium oxide	0.41	-
Total equiv. alkalies as Na_2O	0.49	-
Insoluble residue	0.3	-
Free calcium oxide	1.0	-
Computed compound composition, %		
Tricalcium silicate	53.0	-
Dicalcium silicate	-	-
Tricalcium aluminate	8.3	-
Physical properties		
Specific gravity	-	2.61
Specific surface (Wagner), sq cm/g	1816	-
Specific surface (Blaine), sq cm/g	-	2720
Passing No. 325 sieve, %	-	88.7
Autoclave expansion, %	0.114	-
Time of setting (Gillmore test)		
Initial, hr	3.0	-
Final, hr	6.5	-
Compressive strength (1.2.75 mortar)		
At 3 days, psi	1815	-
At 7 days, psi	2525	-
At 28 days, psi	4000	-
Mortar air content, %	5.0	-

[a] Hawkeye Portland Cement Company, Des Moines, Iowa.

[b] Detroit Edison Company, St. Clair Power Plant, Detroit (St. Clair), Michigan.

of the flyash as measured by the amount passing the No. 325 sieve (6). The St. Clair Power Plant flyash is representative of what presently is considered good quality flyash.

METHODS USED

Cement Requirement

The minimum percentage of portland cement required for each of the soils to meet PCA criteria for soil-cement was determined by the ASTM standard test procedures (ASTM Designations: D558-44, D559-44, D560-44).

Mixing and Molding

Soil-cement-flyash mixes were proportioned and mixed dry; then optimum water content for maximum standard Proctor density was added and the materials were machine-mixed for 4 minutes.

Two-inch diameter by 2-in. high specimens for unconfined compressive strength tests were prepared at approximate standard Proctor density with a double plunger drop-hammer molding apparatus. Four-inch diameter by 4.6-in. high specimens for freezing and thawing tests were prepared with the standard Proctor compaction apparatus.

Curing

The specimens were double wrapped in waxed paper and aluminum foil to better preserve moisture and to prevent entry of carbon dioxide from the air. Curing for periods of 7, 28 and 120 days was done in a moist curing room at 70±3 F and a relative humidity of not less than 90 percent.

Unconfined Compressive Strength Test

At the end of the 7, 28 or 120-day curing periods, 2-in. by 2-in. specimens were unwrapped, immersed in distilled water for 24 hours and then tested for unconfined compressive strength using a load travel rate of 0.10 in. per minute. Tests were run in triplicate and average strengths reported in psi; no correction was made for the ht/diam ratio which was one.

Freezing and Thawing Test

Seven and 28-day cured 4-in. by 4.6-in. specimens were used in the freezing and thawing test which was conducted according to the ASTM standard procedure (ASTM Designation: D560-44) with the following modification: duplicate specimens were used for the loss on brushing measurements, and volume change was not determined.

FLYASH AS AN ADDITIVE TO SOIL-CEMENT

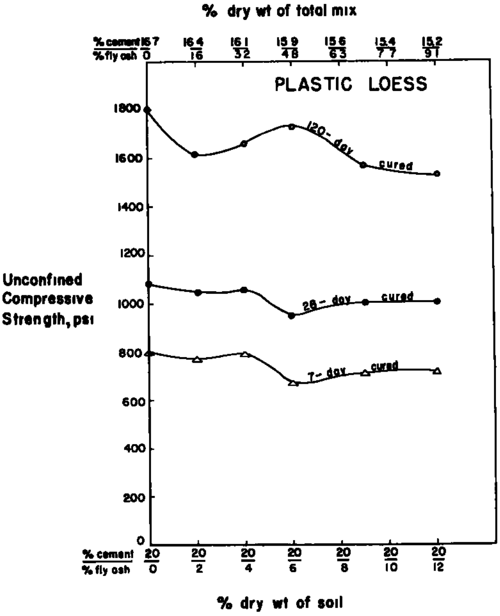

Figure 2. Effect of flyash additives to plastic loess-cement on immersed unconfined compressive strength, when the portland cement content is near the minimum requirement for soil-cement.

As an additive to soil-cement, flyash in the amounts used may be considered mainly a replacement for the soil, with the cement content remaining nearly constant. The extent to which this is true is shown by the upper abscissa scales in Figures 1, 2, 3 and 4, where on a total mix dry weight basis the maximum replacements of cement by flyash are only 0.8 percent (friable loess) to 1.6 percent (alluvial clay). Thus the lower abscissa scales, where both cement and flyash contents are expressed as percentages of the soil dry weight, can be used to obtain an indication of the effect on strength and durability of varying flyash as an additive when the portland cement content is the minimum requirement for soil-cement. The minimum cement requirements were 9 percent for the friable loess, 20 percent for the plastic loess, 21 percent for the alluvial clay and 11 percent for the dune sand, all percentages being of the soil dry weight.

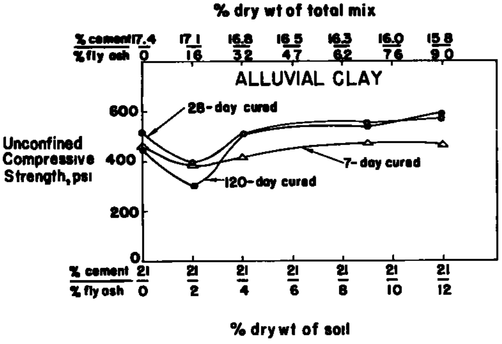

Figure 3. Effect of flyash additives to alluvial clay-cement on immersed unconfined compressive strength, when the portland cement content is near the minimum requirement for soil-cement.

Effect on Unconfined Compressive Strength

Friable Loess. Adding flyash to friable loess-cement resulted in strength gains of about 10 percent after 7 and 28 days curing, but no gain due to flyash was apparent at 120 days (Fig. 1). No additional benefit to strength was obtained with more than 2 percent flyash. The reason for the strength decrease with 4 percent flyash, which occurred at all ages, is not known;

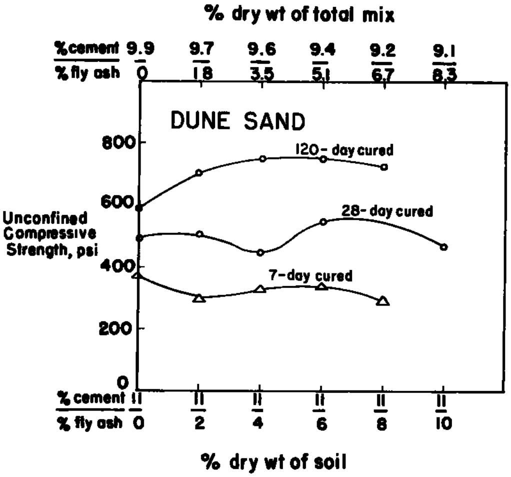

Figure 4. Effect of flyash additives to dune sand-cement on immersed unconfined compressive strength, when the portland cement content is near the minimum requirement for soil-cement.

but the decreases with 10 percent and 12 percent flyash probably reflect decreasin portland cement content (see upper abscissa scale).

The strength gains are attributed to cementation products resulting from the pozzolanic reactions between the flyash and the lime and alkalies liberated by the hydrating portland cement, as discussed previously. With friable loess and the amount of cement used, evidently only a small amount of flyash (about 2 percent) is needed to obtain maximum strength gain from pozzolanic reactions. No other benefits to friable loess-cement from the use of flyash as an additive were observed. Compacted density was not significantly changed by the amounts of flyash used.

Plastic Loess. There was no indication of pozzolanic action in the plastic loess-cement-flyash mixes at 7, 28 or 120 days (Fig. 2). The slight decrease of 7- and 28-day strengths with increasing flyash content may be due either to the lubricating action of the predominantly spherical flyash particles or to the decreasing cement content (see upper abscissa scale) or to both effects. The 120-day strength data generally show the same trend as the 7- and 28-day, but the decrease in strength is considerably

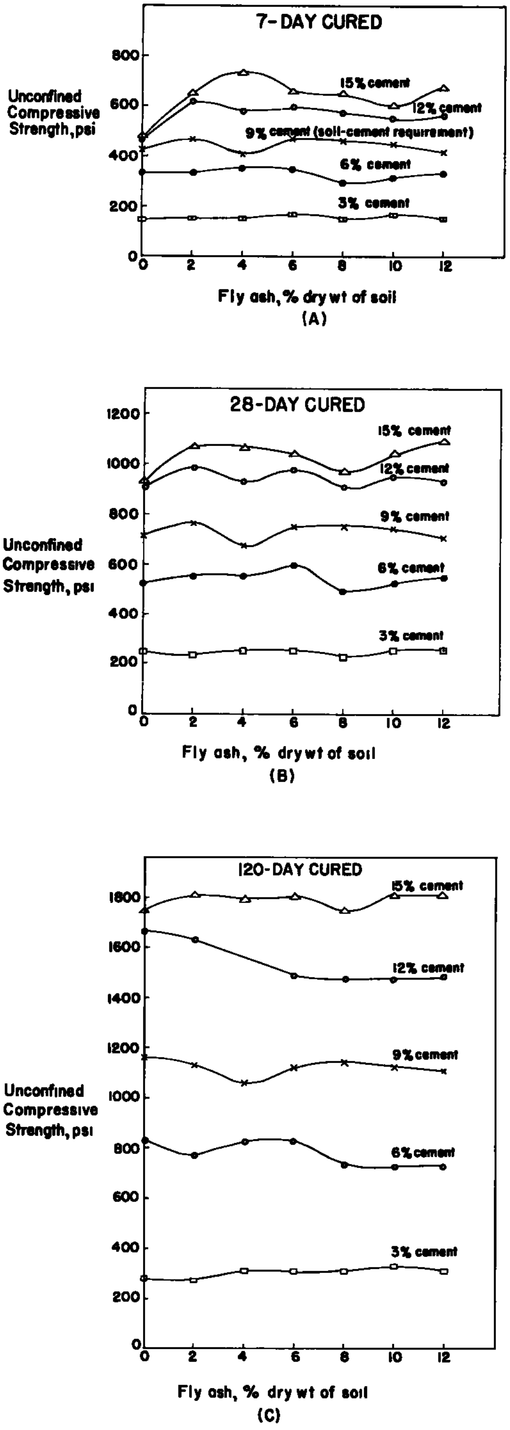

Figure 5. Effect of flyash additives to friable loess-cement on immersed unconfined compressive strength, when the portland cement content is above and below the minimum requirement for soil-cement.

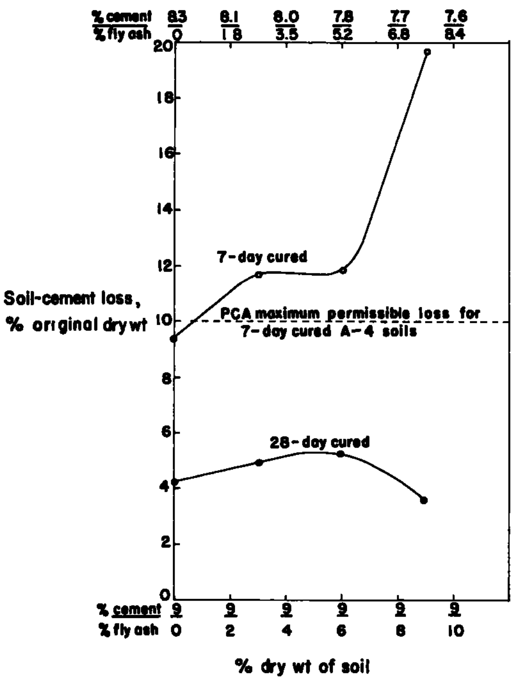

Figure 6. Effect of flyash additives to friable loess-cement on 12-cycle freeze-thaw resistance, when portland cement content is near the minimum requirement for soil-cement.

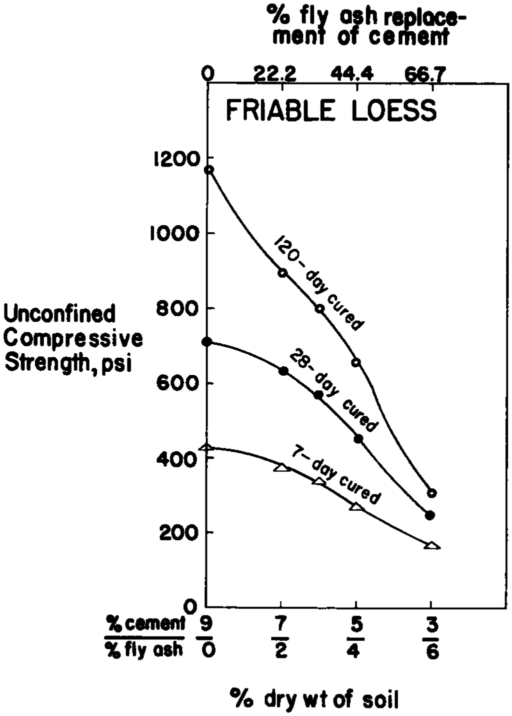

Figure 7. Effect on immersed unconfined compressive strength of partially replacing portland cement with flyash in friable loess-cement.

greater. The reason for the apparent irregularities in the curves is not known; they are not due to variation in compacted density, which was very slight.

Alluvial Clay. The strength gains attributable to flyash additives to alluvial clay-cement are mainly due to reduction of shrinkage during curing (Fig. 3). This benefit was especially evident for 120-day cured specimens; those containing no flyash were badly cracked, which is the reason the zero flyash strength is lower at 120 days than at 7 and 28 days. Four percent or more of flyash reduced cracking of 120-day cured specimens, and those containing 9 percent and 12 percent flyash showed no surface cracks. As in the case of friable loess and plastic loess, the flyash additives to alluvial clay-cement did not cause significant variation in compacted density.

Dune Sand. The most encouraging results with flyash as an additive were obtained with dune sand-cement (Fig. 4). At 120 days the strength with 4 percent to 6 percent flyash was about 28 percent greater than the strength without flyash. The beneficial effects of flyash in the dune sand-cement are attributed to two related factors: first, the flyash acted as a filler—6 percent increased the compacted density from 114.9 to 117.2 pcf; second, the flyash acted as a pozzolan and the resulting cementation was made more effective by the improvement in grain contact areas. At 7 days the lubricating quality of the flyash evidently overshadowed the beneficial effects, resulting in a net decrease in strength, but at 28 days the benefits appear and continue to increase up to 120 days.

Comparison of Benefits. The different results obtained with the four soils indicate that soil texture has an important effect on the benefits from flyash as an additive in soil-cement. Although this investigation was not extensive enough to obtain a correlation between beneficial pozzolanic activity and the clay content of soil, there is

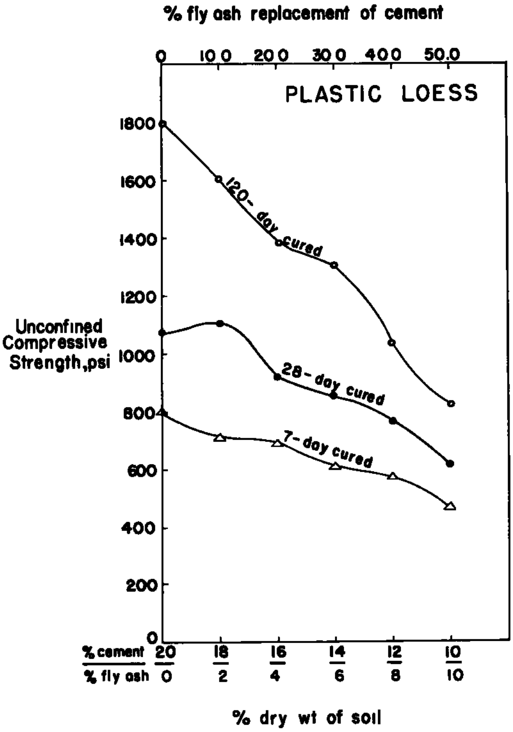

Figure 8. Effect on immersed unconfined compressive strength of partially replacing portland cement with flyash in plastic loess-cement.

evidence that such a relationship exists. In the two high clay content soils, plastic loess (41.8 percent 5 micron clay) and alluvial clay (74.3 percent 5 micron clay), there was no sign of beneficial pozzolanic reactions between flyash and hydration products of portland cement, presumably because of clay coatings on the flyash or fixation of lime by clay or a combination of both effects. In the friable loess (17 percent 5 micron clay) and in the dune sand (3 percent 5 micron clay), sufficient lime and alkalies from the hydrating portland cement were available to react with from 2 to 6 percent flyash; the best strength gains to pozzolanic reactions were obtained with the dune sand, probably because of its low clay content.

The other beneficial effects of flyash additives in soil-cement, reduction of shrinkage cracking in the alluvial clay mixes and improvement of gradation in the dune sand mixes, are also important, particularly to ultimate strength.

Influence of Cement Content. The friable loess was used to check on the beneficial effect of flyash additives when the portland cement content is above and below that required for soil-cement, in this case 9 percent. As shown in Figure 5A, at 7 days there is definite indication of increased pozzolanic action for cement contents higher than 9 percent: for 15 percent cement the strength gain due to flyash reached a maximum of 53 percent with 4 percent flyash, as compared to the 40 percent gain for 12 percent cement with 2 percent flyash, and the 10 percent gain for 9 percent cement with 2 percent flyash. For cement contents less than 9 percent there was little or no indication of pozzolanic activity. Thus it appears that with the friable loess and the Type I portland cement used, at least 9 percent cement is needed to provid sufficient lime and alkalies for significant pozzolanic reactions with flyash at 7 days. The use of larger amounts of cement resulted in the formation of more reaction products as evidenced by the greater strength gains.

The 28-day strength curves (Fig. 5B) generally display the same trends as the 7-day curves, but the strength gains from pozzolanic action are less. At 120 days (Fig. 5C) there is little or no remaining evidence of benefit to strength from pozzolanic action. The apparent decrease of pozzolanic activity with increased curing time is contrary to expectations; it will be recalled that the opposite trend was obtained in the experiments with dune sand. It may be that there was no re-

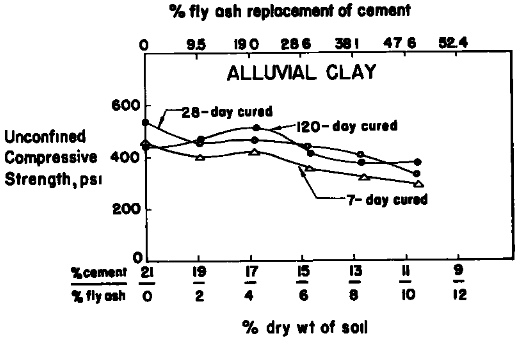

Figure 9. Effect on immersed unconfined compressive strength of partially replacing portland cement with flyash in alluvial clay-cement.

duction of pozzolanic activity at 28 and 120 days, as would appear from the data, but rather that there was interference of some sort with the normal portland cement hydration process. This of course is just a guess.

Effect on Freeze-Thaw Resistance

Although it is known that a relationship exists between the unconfined compressive strength and freeze-thaw resistance of soil-cement, there is always the possibility that an additive to soil-cement, such as flyash, will produce unexpected results. As a check on this, the effect of flyash on the freeze-thaw resistance of soil-cement was studied with the friable loess. To save on testing time, only the soil-cement weight-loss-with-brushing part of the standard freeze-thaw test procedure was conducted.

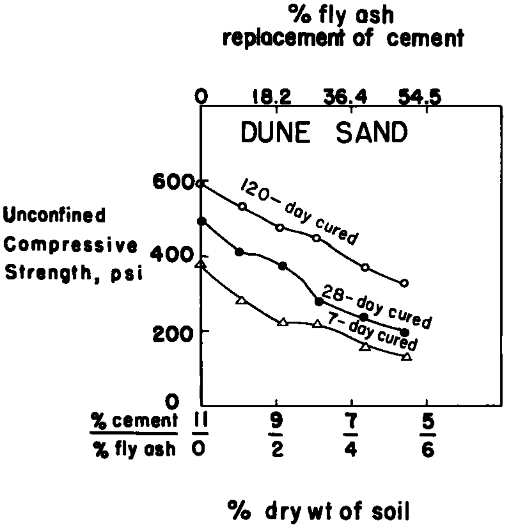

Figure 10. Effect on immersed unconfined compressive strength of partially replacing portland cement with flyash in dune sand-cement.

According to the PCA criteria for soil-cement, the maximum permissible friable loess-cement loss in weight during 12 cycles of freezing and thawing, with brushing after each cycle, is 10 percent of the original dry weight of the test specimen. As shown in Figure 6, the addition of flyash to friable loess-cement increased the 12-cycle weight losses of 7-day cured specimens to above the maximum permissible value; flyash contents greater than 6 percent appear especially detrimental, probably because of the lubricating action of the flyash and the consequent lowered resistance to brushing.

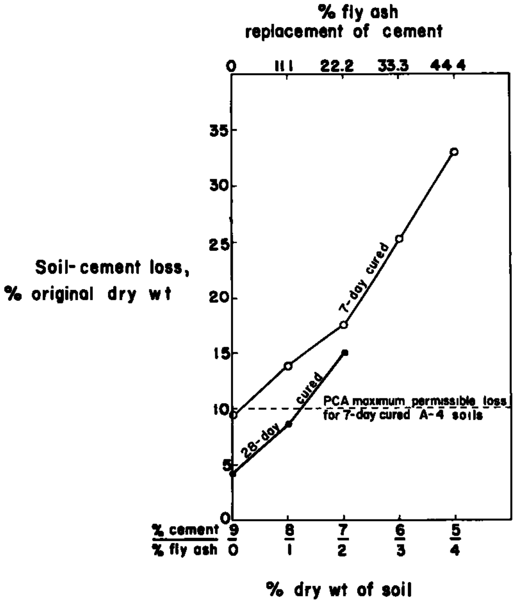

Figure 11. Effect on 12-cycle freeze-thaw resistance of partially replacing portland cement with flyash in friable loess-cement.

If the interpretation of a lubrication effect is correct, it would mean that the brushing test is relatively more severe for soil-cement-flyash than for pure soil-cement, and the results should be weighed accordingly. Brushing of course does not correspond to any expected field usage of soil-cement-flyash in base course construction, since resistance to abrasion is not required.

The weight losses of 28-day cured specimens, both with and without flyash, were considerably lower than the 7-day maximum allowable loss. Also, on the basis of the 28-day data, flyash additives to friable loess-cement appear to be much less detrimental to freeze-thaw resistance, possibly because the increased pozzolanic activity with longer curing compensated for some of the flyash lubrication effects.

FLYASH AS A PARTIAL REPLACEMENT FOR PORTLAND CEMENT

This part of the investigation was made to determine the feasibility of using flyash as a partial replacement or substitute for portland cement in soil-cement. The replacement of cement with flyash is expressed in two different ways in Figures 7, 8, 9, 10 and 11. The bottom abscissa scales show the relative percentages of cement and flyash on the basis of the dry weight of the soil; the top scales express the flyash replacement of cement as a percentage of the cement requirement for soil-cement.

Effect on Unconfined Compressive Strength

Even a cursory examination of the data trends in Figures 7, 8, 9 and 10 shows that cementation from pozzolanic reaction products does not compensate for that lost due to the reduction of portland cement content. Apparently not enough lime and alkalies are available for beneficial pozzolanic reactions with flyash when the cement content is below that for soil-cement; this conclusion is also supported by the previously discussed data in Figure 5. It is possible that replacements smaller than those tried might cause only slight loss of strength, but this possibility is of little practical interest, since it seems doubtful that flyash would be used in the field in quantities less than 2 percent of the dry soil weight.

The only definite benefit from flyash as a replacement for portland cement was observed with the 120-day cured alluvial clay-cement-flyash specimens which showed an increase in strength due to less shrinkage cracking (Fig. 9).

Effect on Freeze-Thaw Resistance

The question naturally arises, how strong does soil-cement have to be? Maybe some strength reduction is permissible providing resistance to freezing and thawing is satisfactory.

The answer for one soil, the friable loess, is evident from Figure 11. Freeze-thaw resistance at both 7 and 28 days is drastically decreased by even small replacements of cement by flyash.

CONCLUSION

In conclusion it would seem that for use in soil stabilization with portland cement, flyash is more promising as an additive than as a replacement for part of the portland cement.

The greatest benefits from flyash as an additive in soil-cement appear to be obtainable with poorly graded, low clay content soils such as dune sand, in which improvement in strength is the result of more surface contact areas and complimentary cementation from pozzolanic reaction products.

Flyash, both as an additive and as a partial replacement for cement, was observed to reduce shrinkage cracking during curing of portland cement stabilized highly plastic clay soil. Whether sand or other relatively inert material would have been equally effective was not determined.

ACKNOWLEDGMENT

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REFERENCES

1. Portland Cement Association, "Soil-Cement Laboratory Handbook." Chicago, The Association (1956).

2. American Society for Testing Materials, "Book of ASTM Standards." Part 3,

p. 201, Philadelphia, The Society (1955).

3. Blanks, R. L., "Fly Ash as a Pozzolan." Journal, Am. Concrete Inst. 21(No. 9): 701-707 (1950).

4. Davis, R. E., Davis, H. E. and Kelly, J. W., "Weathering Resistance of Concrete Containing Fly-Ash Cements." Journal, Am. Concrete Inst. 12(No. 3): 281-293 (1949).

5. Timms, A. G. and Grieb, W. E. "Use of Fly Ash in Concrete." Proceedings, Am. Soc. Testing Mats. 56: 1139-1160 (1956).

6. Brink, R. H. and Halstead, W. J., "Studies Relating to the Testing of Fly Ash for Use in Concrete." Proceedings, Am. Soc. Testing Mats., 56: 1161-1214 (1956).

7. Baker, R. F., Discussion. In Minnick, L. J. and Miller, R. H. "Lime-Fly Ash Compositions for Use in Highway Construction." pp. 498-499, Proceedings, HRB 30: 489-502 (1950).

8. U.S. Public Roads Administration, "Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft." Washington, D.C., U.S. Government Printing Office (1943).

9. Davis, R. E., "A Review of Pozzolanic Materials and Their Use in Concrete." Am. Soc. Testing Mats., Symposium on Pozzolanic Materials in Mortars and Concrete. ASTM STP No. 99, p. 3, Philadelphia, The Society (1949).

Discussion

L. T. NORLING, Soil-Cement Bureau, Portland Cement Association, Chicago — Davidson's paper adds much needed information on the effect of adding flyash to soil-cement mixtures.

It is significant that no beneficial pozzolanic reaction due to addition of flyash as an admixture was obtained for the plastic loess and alluvial clay, and only slight benefit was obtained in some instances for the friable loess soil. Similar results were reported by A. A. Lilley in the Cement and Concrete Association, London, England, Technical Report TRA/158, October, 1954. Lilley reports that flyash does not contribute to the strength of a cohesive soil or soil-cement mixture. The compressive strengths were proportional to the percentage of cement in the mixture.

Some increase in compressive strength, particularly after 120-day cure, was obtained by Davidson for a dune sand soil. As he points out, the benefit is due to the flyash acting as a filler and possibly to some pozzolanic action. It may be pointed out that the addition of a friable silty or clayey soil to the dune sand would also act as a filler and may increase the compressive strength to the same degree as the flyash admixture did. On most soil-cement projects the friable borrow soil could be obtained at less cost than the flyash.

The reduced resistance of the friable loess to freezing-and-thawing reported by Davidson is particularly significant. Exploratory tests run in the Portland Cement Association Soil-Cement Laboratory gave similar results. For example, the addition of 6.2 percent flyash by weight of soil to a clay soil-cement mixture (PCA Soil No. 5787) almost doubled the losses due to freezing and thawing. In similar tests on a gravelly soil-cement mixture (PCA Soil No. 5773), losses were doubled by the addition of 2.5 percent flyash by weight of soil. In both these instances the higher losses were due to softening of the specimen and not to abrasion of the brush.

Davidson's paper shows the detrimental effects of adding flyash as a partial replacement for cement in soil-cement work. Compressive strengths were lowered appreciably for each of the four soils tested and the resistance of the friable loess to freezing and thawing was decreased drastically. (Freeze-thaw tests were not run on the other three soils.) This is in agreement with the work reported by Lilley and with exploratory tests run in the Portland Cement Association's Soil-Cement Laboratory.

In summary, the data in Davidson's paper show that flyash cannot be used as a partial replacement for cement in soil-cement. Compressive strengths were lowered and the resistance to freezing and thawing was reduced drastically. The data show that the addition of flyash as an admixture may be practical or economical in soil-cement work where a filler is beneficial such as in the case of dune sands. However, benefit may also be obtained by adding a friable silty or clayey soil as a filler or by adding a small additional amount of cement.

Study of Soil-Cement Base Courses on Military Airfields

J. F. REDUS, Engineer, Flexible Pavement Branch, Soils Division, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

A group of military airfields with pavements composed of soil-cement base courses and asphaltic-concrete surfacing was studied. Laboratory tests were performed on samples of the soil-cement from four fields; observations were made on the remaining fields. The natural soils used in the soil-cement ranged from beach sand to lean clay and traffic varied from very light training planes to cargo craft.

The soil-cement performed reasonably well under the imposed loads and showed no signs of deterioration with age. The investigation also indicated that small percentages of cement tended to reduce the plasticity in some borderline base-course materials and aided construction with some rounded cohesionless sands.

● A NUMBER of military airfields in the United States have been constructed with soil-cement base courses. An investigation of several of these fields was made by the Waterways Experiment Station to determine the behavior of the base courses under the conditions to which they had been subjected. This paper presents the results of the investigation. Studies were made at the following airfields where a portion of the paved facilities were constructed with soil-cement base courses: Turner AFB, Albany, Georgia; Moody AFB, Valdosta, Georgia; West Palm Beach AFB, West Palm Beach, Florida; Hot Springs, Arkansas, Municipal Airport and Adams Field (Little Rock, Arkansas, Municipal Airport); Clovis AFB, Clovis, New Mexico; and Edwards AFB, Muroc, California. Detailed visual inspections were made of all the pavements, and field and laboratory tests were performed at West Palm Beach, Moody, and Turner AFB's and at Hot Springs Municipal Airport.

INVESTIGATION

The soil, climate, and traffic varied considerably for the fields investigated. In order that these differences may be clearly understood, a brief description of each field is given in the following paragraphs.

Turner AFB

This field is located near Albany, Georgia, in the East Gulf Section of the Gulf Coastal Plain. The climate is warm and humid. Natural soils at the site ranged from nonplastic silty sands to clayey sands with maximum plasticity index values of about 20. The 6-in. base course on three runways and connecting taxiways was constructed of the natural soils, nonplastic and plastic, mixed with about 10 percent cement. The pavements were constructed in the period 1941 to 1943. The asphaltic-concrete wearing course was placed about nine months after the base course was completed. For six years after construction, the pavements were used by planes with about 12,000-lb wheel loads and tire pressures of about 65 psi. In 1951 and 1952, planes with 7,000- to 10,000-lb wheel loads and tire pressures of about 200 psi used the field. Observations and tests were made in late 1952.

Moody AFB

Moody AFB is near Valdosta, Georgia, and is also in the East Gulf Section of the Gulf Coastal Plain. The climate here, too, is warm and humid. The natural materials were silty and clayey sands, ranging from nonplastic to plastic with a plasticity index of 10. In 1951, a part of the east N-S runway was constructed with a soil-cement base course, 8 to 10 in. thick, consisting of natural materials and about 10 percent cement. The area was paved with 1½ in. of asphaltic concrete about a year later. The pavement received intense traffic from planes with about 7,500-lb wheel loads and tire pressures of about 200 psi and also from planes with 16,000- to 24,000-lb wheel loads and tire pressures of about 65 psi. The facilities were open to traffic both before and after the asphaltic concrete was laid. Observations and tests were made in late 1952.

West Palm Beach AFB

This field is situated on typical beach sands at West Palm Beach, Florida, in an area of subtropical climate. The 6-in. base course of a parking apron was constructed of soil-cement during 1934-1937. The sands in the mix contain fragments of shell and shell marl. The percentage of cement used is not known exactly, but is believed to be above 10. Asphaltic concrete was placed over the base course in 1942. The pavement has been subjected to traffic since the construction of the base course. Prior to 1942, traffic was composed mainly of very small planes; after 1942, planes with assembly loads ranging from 24,000 to 87,000 lb and tire pressures of about 75 psi used the field extensively. Planes with assembly loads of 80,000 lb and tire pressures of about 175 psi also used the pavements extensively. Tests were made in late 1952; last observations were made in early 1957.

Hot Springs Municipal Airport

Hot Springs Municipal Airport is located in the Ouachita Mountain Section near Hot Springs, Arkansas, in the area of the Novaculite Uplift. The climate is warm and moderately humid. Lean clays and shales are typical natural soil of the site. The base course and asphaltic-concrete pavement were constructed in 1943. The untreated base course material was a crushed stone with binder of natural lean clay with some plasticity. About 5 percent cement was added to the 6-in. base course on portions of the runways, taxiways, and apron to reduce the plasticity index to an acceptable value. Traffic has been composed of a few cycles per day of DC-3 (12,000-lb wheel load) and smaller planes, all with tire pressures of 60 psi or less. Observations and tests were made in early 1953.

Adams Field

Adams Field is located in the Arkansas Valley near Little Rock, Arkansas, which also has a warm, moderately humid climate. The natural material at this site is a nonplastic silty sand which is an alluvial deposit of the Arkansas River. About 10 percent cement was added to this soil to form the base course of a parking apron. The 6-in. soil-cement base course and asphaltic-concrete pavement were constructed in 1941. Traffic has been composed of a few light training and passenger planes. Observations were made in 1952.

Clovis AFB

Clovis AFB is located near Clovis, New Mexico, in the physiographic area known as the Llano Estacado. The climate is warm and semi-arid. The natural soil at the field is a lean clay, but caliche with some plasticity was obtained from nearby sources and mixed with about 10 percent cement to form a soil-cement base course about 6 in. thick for the east N-S runway. The base course was completed and a bituminous seal placed in early 1954. Traffic has consisted mostly of planes with 12,000-lb wheel loads and tire pressures of about 60 psi. Observations were made in the fall of 1954.

TABLE 1

Field	Pavement Age Years	Percentage of Cement	Natural Soil		Soil-cement		Percentage Loss after 12 Cycles, Brushed		Maximum Allowable* Loss, %
			LL	PI	LL	PI	Wetting-drying	Freezing-thawing	
Turner	10	10	29	17	--	NP	12.9	35.8	14
Moody	1	8	25	12	--	NP	9.2	11.5	14
West Palm Beach	15	10+	--	NP	--	NP	**	24.7	14
Hot Springs	9	5	27	12	--	NP	---	---	14

* Not a standard, but recommended by PCA.
** Specimens disintegrated after seven cycles.

Edwards AFB

This field is located near Muroc, California, in the Mojave Desert along the west shore line of Rogers Dry Lake. The climate is hot and dry. The natural soils at this site range from nonplastic silty sands to plastic clayey sands with plasticity index of 8 to 12. The base course for certain streets and drives at the field were constructed of a mixture of mostly nonplastic silty sand and about 10 percent cement. Part of the base course was completed in 1942 and was paved with asphaltic concrete in 1943; the remainder was constructed and paved in 1953. All traffic was vehicular, but included appreciable amounts of moderate-weight trucks. Observations were made in 1954.

TEST DATA

The pavements on each field were observed very carefully to determine the surface conditions. In-place tests were performed at Turner, Moody and West Palm Beach AFB's and Hot Springs Municipal Airport, and samples were obtained for laboratory testing. In-place tests included determination of moisture content and density on the base course and CBR on the subgrade, and the laboratory work included Atterberg limits, freezing-thawing, and wetting-drying tests. In-place CBR tests were considered inapplicable to soil-cement because of its hard, brittle nature. Results of the laboratory tests are summarized in Table 1.

ANALYSIS

Behavior Under Traffic

Observations of the surface conditions of the various pavements revealed cracks in all the pavements. The cracks usually formed rectangular patterns of varying size (Fig. 1) and occurred in areas untouched by traffic as well as traveled areas. The distribution and spacing of cracks varied somewhat over each field, but generally the condition was the same on all the fields. The age of the soil-cement base course at the time of placement of the bituminous paving varied from a few months to eight years, but in each case cracks appeared within a few months after pavement was placed. No measurable permanent deformation was found at any of the cracks or in any other areas of the fields. It appears that the cracking was caused by shrinkage of the soil-cement and was not connected with overload.

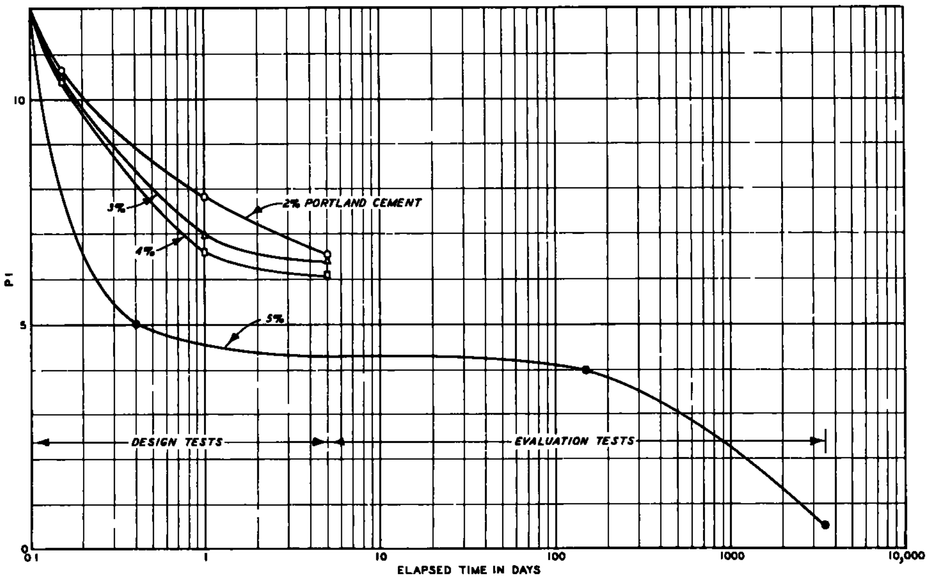
Accelerated traffic tests conducted by the Corps of Engineers showed that the soil-cement base course along construction joints did not afford the same protection to the underlying layers as that in the interior of construction lanes (1). No evidence of lack of protection at the joints was noted at any of the fields; however, all but one of the fields were capable of carrying heavier planes than those in use, as shown in the following analysis. In-place CBR values for subgrades at Turner, Moody, and West Palm Beach AFB's and at Hot Springs Municipal Airport were 15, 15, 20 and 15, respectively. Experience has shown that sands such as found at West Palm Beach develop high strengths when confined, although the high values may not be measured by the CBR test. Thus, the CBR value of 20 is believed very conservative for this field. Hot Springs Municipal Airport received so little traffic that it is not considered as having been proved satisfactory, although no distress developed. Using the subgrade CBR value of 15 and thickness above the subgrade of 8 in. for Turner AFB, the pavement would be adequate for unlimited use by planes with assembly loads of 12,000 lb and single wheels with either low- or high-pressure tires. Based on a subgrade CBR of 15 and a thickness above the subgrade of 10 in., the pavement at Moody AFB would be satisfactory for unlimited use by planes with 19,000-lb load on single wheels with low-pressure tires, or planes with 16,000-lb load on single wheels with high-pressure tires. The pavements at Moody AFB would probably carry wheel loads up to 25,000 lb for one or two years, but could not support an imposed wheel load of 24,000 lb for an unlimited period.

Figure 1. Typical cracking pattern.

The Corps of Engineers also found poor bond between pavement and soil-cement at Beltsville, Maryland (2), and Barksdale, Louisiana (3), and noted slippage of the pavement in some of these areas. Good bond between pavement and base was found on all the fields tested in this study, and observations revealed no evidence of slippage on any of the pavements.

Wetting-Drying, Freezing-Thawing Tests

The results of 12 cycles of wetting-drying and freezing-thawing tests on samples of soil-cement from three of the four fields tested are shown in Table 1. There is no



PLASTICITY INDEX VS TIME
SOIL-CEMENT MIXES
HOT SPRINGS, ARK

standard specification for the maximum allowable material loss, but that recommended by the Portland Cement Association for each of these tests is 14 percent. The loss from specimens from West Palm Beach AFB exceeded 14 percent in the wetting-drying tests, and the loss from specimens from Turner and West Palm Beach AFB's exceeded 14 percent in the freezing-thawing tests. Since none of these fields are in areas of appreciable frost penetration, prototype behavior cannot be compared with that predicted by the freezing-thawing tests. However, presumably all were subject to wetting and drying, and it is considered significant that materials from one of the fields failed the wetting-drying test but behaved satisfactorily in the prototype.

Reduction in Plasticity

For many years the plasticity index has been used as an indicator of base course quality; construction specifications usually contain a maximum allowable value. These maximum allowable values have varied considerably since they are usually based on local experience, but it has been generally agreed that materials with low plasticity or nonplastic materials are more satisfactory than those of higher plasticity. On this basis, it is believed that the plasticity index is one of the more important indicators of soil-cement base course quality.

The data in Table 1 show that those materials having plastic binders at the time of construction were nonplastic when tested one to ten years later. A study of the effects of cement on plasticity index with time can be made from data available from Hot Springs. In arriving at the figure of 5 percent cement to be used in the base course mix, a laboratory study was made using percentages of cement varying from 2 to 5. Atterberg limits tests were performed in connection with the laboratory design work at the end of a few hours, one day, and five days; and tests were performed on samples of the base course at 150 and 3,500 days after construction. The results of these tests are shown in the plot of plasticity index values versus time for varying percentages of cement on Figure 2. These data indicate that all percentages of cement reduced the plasticity index of the material considerably in the first 24 hours, the amount of reduction increasing as the percentage of cement increased. Results of tests at 150 and 3,500 days indicate that the plasticity index of the mixture with 5 percent cement continued to decrease. Many similar instances of successful use of small percentages of cement (usually less than 5) to reduce the plasticity of a borderline base course material to a satisfactory value are cited in highway literature.

Healing of Cracks

Examination of samples of soil-cement from Moody and West Palm Beach AFB's showed that numerous cracks in the material had been filled by a deposit of calcareous material. The specimens showed no tendency to break along these cracks; on the contrary, they usually broke in uncracked portions.

Compaction

In-place density was determined at the four fields tested, but laboratory curves were not available from which the degree of compaction could be determined. However, knowledge of the compaction practices at the time the older pavements were constructed and of the specified compaction for the newer pavements suggests that the degree of compaction probably ranged from 85 to 95 percent of modified AASHO maximum density. Since the behavior of the soil-cement was the same on all the fields and since the wheel loads and intensity of traffic varied rather widely, it is indicated that any lack of compaction must have been adequately compensated for by cementation

DISCUSSION

The investigations indicated that cracks in the various soil-cement bases were caused by factors other than the imposed loads. The amount of cracking did not vary noticeably from field to field and cannot be related to type of material. The only explanation that seems reasonable is that the cracking is caused by shrinkage of the soil-cement. The presence of cracks appeared to have no effect on the ability of the soil-cement to carry the imposed loads or on the durability of the soil-cement mixture. The thickness of pavement and base and the quality of base at Turner AFB appeared to be adequate for indefinite operation of the imposed loads; those at Moody AFB appeared to be adequate for at least limmited operation of the imposed load. No comparison between construction joints and interiors of construction lanes can be made from this study since all were equally satisfactory. The bases provided adequate protection for the subgrades in all areas and were structurally adequate within themselves. Absence of pavement slippage appeared to be due to the good bonds between base and pavement noted on these fields.

The wetting-drying tests did not satisfactorily predict the behavior of the soil-cement base courses at West Palm Beach, but did indicate that the base courses at Turner and Moody AFB's would be satisfactory. Therefore, this test may not be indicative of the behavior of the mixture when the natural material is a beach sand.

The Atterberg limits tests indicate that cement reduced the plasticity index of the various plastic materials and that the reduction has been permanent. The amount of reduction varied directly with the percentage of cement used; the time required to produce a given reduction varied inversely with the percentage of cement used when the percentages were 5 or less.

No method was found to evaluate the effects of the healing of cracks by deposition of a calcareous material. It is not known whether the material was deposited before or after the cracks occured in the asphaltic concrete. However, from observations of the healing, it is strongly suspected that the calcareous material was deposited after the original crack appeared since the crack seems well cemented by the deposit.

SUMMARY

The following summarizing statements are believed warranted:

a. Cracks occurred in the soil-cement base courses on all the fields, but appeared to have no connection with traffic.

b. For the traffic imposed, the soil-cement base courses were structurally adequate and produced satisfactory protection for the subgrade materials.

c. The results of wetting-drying tests did not corrèlate with prototype behavior in every case.

d. The plasticity index of materials with excessive plasticity was reduced to values at or near zero by the addition of 5 to 10 percent cement.

e. The reduction in plasticity appears to be permanent, and durability of the soil-cement is satisfactory in all cases.

REFERENCES

1. Boyd, W. K., "Accelerated Traffic Tests for Flexible Airfield Pavements." Proceedings, HRB (1946).
2. Corps of Engineers, Waterways Experiment Station, "Summary Report, Accelerated Traffic Tests." Vicksburg, Miss. (February, 1947).
3. Corps of Engineers, U.S. Army, Little Rock District, "Service Behavior Test Section, Barksdale Field, La." Little Rock, Ark. (July, 1944).

Discussion

E. G. ROBBINS, Soil-Cement Bureau, Portland Cement Association, Chicago—Mr. Redus' study of soil-cement paving confirms the observation that reflection cracking of a soil-cement base through the bituminous surface is not associated with structural failure, but rather that cracking is a function of the drying and hardening of the base.

Calcareous deposits have also been observed in cracks and around aggregate in freshly broken soil-cement samples. Laboratory tests have demonstrated the "healing" ability of soil-cement. Specimens fractured in compression testing after 7-days moist curing and then returned to the moist room and rebroken at 28 days, have had compressive strengths only 10 percent lower than the strengths of specimens cured continuously 28 days in the moist room.

Wet-dry and freeze-thaw tests are empirical laboratory procedures to determine how much cement is required to harden a soil adequately. For specimens made of sandy soils, the allowable 14 percent loss resulting from 12 cycles of testing is a criterion based on the results of thousands of tests and on the effects of weathering on laboratory specimens subjected to outdoor exposure. It was not unusual for losses resulting from 12 cycles of wet-dry and freeze-thaw testing of cores taken from pavements to be higher than losses observed in tests of laboratory specimens. The tests and criteria, however, have been successfully used to build many thousands of miles of roads and, as Mr. Redus indicates, many airports which are giving good service. It is not unusual for laboratory soil-cement specimens made of dune or blow sand to have higher losses in the wet-dry test than in the freeze-thaw test.

Tests also show that the effect of cement in reducing the plasticity index (PI) of a soil continues for a long period of time, although most of the reduction occurs in the first 3 to 7 days. For example, a clay with a PI of 14 was treated with 7 percent cement. Immediately after construction, the PI had dropped to 11, and after 6 years the material had PI's of 5 to 11.

Most highway specifications require compaction of at least 95 percent of standard AASHO density. This requirement is readily attained during construction. As a soil-cement base cures, the cement hardens the material in this dense state, and there is no appreciable further densification under traffic. Only enough initial compaction is required to bring the soil grains into intimate contact with the cement. Higher densities, however, are beneficial to the quality of soil-cement provided enough water is available for cement hydration.

Expanded Short-Cut Test Method for Determining Cement Factors for Sandy Soils

L. T. NORLING, Laboratory Chief, and
R. G. PACKARD, Assistant Laboratory Chief,
Soil-Cement Bureau, Portland Cement Assoiciation

The short-cut testing procedures are presented as two methods. Method A is used for sandy soils having no material retained on the No. 4 sieve. Method B, newly developed, is used for sandy soils containing material retained on the No. 4 sieve. Comparisons of data have verified the accuracy and dependability of the newly developed charts used with Method B. Data show that Method B provided safe cement factors for 204 (97.6 percent) of the 209 soils studied.

●IN 1952, THE PORTLAND CEMENT ASSOCIATION developed a short-cut method for determining cement factors for soil-cement construction using sandy soils. The method evolved from a correlation of data obtained from testing 2,229 sandy soils using complete ASTM or AASHO test procedures. The original short-cut test method was the subject of the Highway Research Board papers presented in January 1953 (1) and in January 1955 (2).

The short-cut test method does not involve new tests or additional equipment. Instead, it is possible to eliminate some tests by using charts developed from previous tests on similar soils. Thus, the only tests required are a grain size analysis, a moisture-density test and compressive strength tests. Relatively small samples are needed. All tests, except for the 7-day compressive strength tests, may be completed in one day.

The original charts for the short-cut test method were based on the fraction of the soil passing a No. 4 sieve. This was because the data then available for correlation were based on tests of specimens designed on that fraction. To use this method with soils containing material retained on the No. 4 sieve, it was necessary first to determine, by charts, the cement requirement of the soil fraction passing the No. 4 sieve and then, for field construction, to calculate the cement requirement of the total mixture. Also, the maximum density and optimum moisture content of the total soil-cement mixture were calculated for field construction based on tests on the fraction passing the No. 4 sieve.

It is important to determine the maximum density and optimum moisture content of a mixture that represents, as nearly as possible, the material to be used in building soil-cement. Therefore, for several years, the Portland Cement Association and others have run the moisture-density test and designed soil-cement test specimens on total material (2). This reduces considerably the calculations needed for soils containing material retained on the No. 4 sieve. Further, it results in a maximum density more easily duplicated in test specimens. In 1957 this procedure was adopted as standard by the American Society for Testing Materials and bears ASTM Designation D558-57 (3).

The object of the work reported in this paper was to expand the short-cut method to permit the use of data from tests on the total soil-cement mixture. New charts and recommended procedures were developed for soils containing material retained on the No. 4 sieve. This procedure is referred to in this paper as Method B. The original charts and procedures (Method A) are still used for soils having no material retained on the No. 4 sieve.

The short-cut test procedures do not always indicate the minimum cement factor that can be used with a particular sandy soil. However, they almost always provide a safe cement factor generally close to that indicated by standard ASTM-AASHO wet-dry and freeze-thaw tests. The procedure should prove even more accurate if test data from more localized areas are used to verify the charts.

The short-cut test method is coming into wider use by engineers and testing laboratories handling soil-cement projects. In time, the method may largely replace the standard tests as experience with it grows and the procedures are verified for local sandy soils.

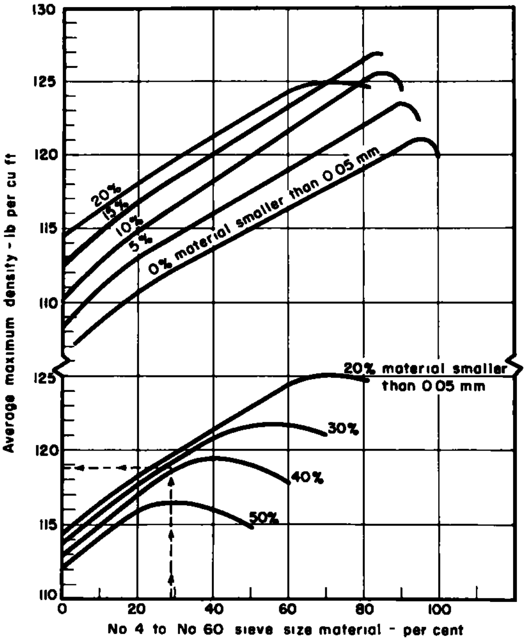


Figure 1. Average maximum densities of soil-cement mixtures having no material retained on No. 4 sieve.

PROCEDURES FOR DETERMINING CEMENT FACTORS

Two methods have been developed for establishing cement requirements for many sandy soils. Method A is used for soils having no material retained on the No. 4 sieve. The new procedure, Method B, is used for soils containing material retained on the No. 4 sieve.

The short-cut method may be applied only to soils containing less than 50 percent material smaller than 0.05 mm (silt and clay) and less than 20 percent material smaller than 0.005 mm (clay). These were the gradation limits for the soils included in the correlation used to develop the original charts. Dark gray to black soils with appreciable amounts of organic impurities were not incleded in the correlation and therefore may not be tested by these procedures. This is also true of miscellaneous granular materials such as cinders, caliche, chat, chert, marl, red dog, scoria, shale, and slag. Moreover, the short-cut procedures do not apply to granular soils having material retained on the No. 4 sieve if that material has a bulk specific gravity less than 2.45.

Before applying the short-cut procedures, it is necessary (a) to determine the gradation of the soil, and (b) to determine the bulk specific gravity of the material retained on the No. 4 sieve to see if it meets the above requirements. If a the soil passes the No. 4 sieve, Method A should be used. If materials is retained on the No. 4 sieve, Method B is used.

Method A

Step 1: Determine by test the maximum density and optimum moisture content for a mixture of the soil and portland cement.[1]

Note 1: Use Figure 1 to obtain an estimated maximum density of the soil-cement mixture being tested. This estimated maximum density and the percentage of material smaller than 0.05 mm (No. 270 sieve) may be used with Figure 2 to determine the cement content by weight to use for the test.

Step 2 : Use the maximum density obtained by test in Step 1 to determine from Figure 2 the indicated cement requirement.

Step 3: Use the indicated cement factor obtained in Step 2 to mold compressive

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[1] "Method of Test for Moisture-Density Relations of Soil-Cement Mixtures," ASTM Designation D558; AASHO Designation T134.

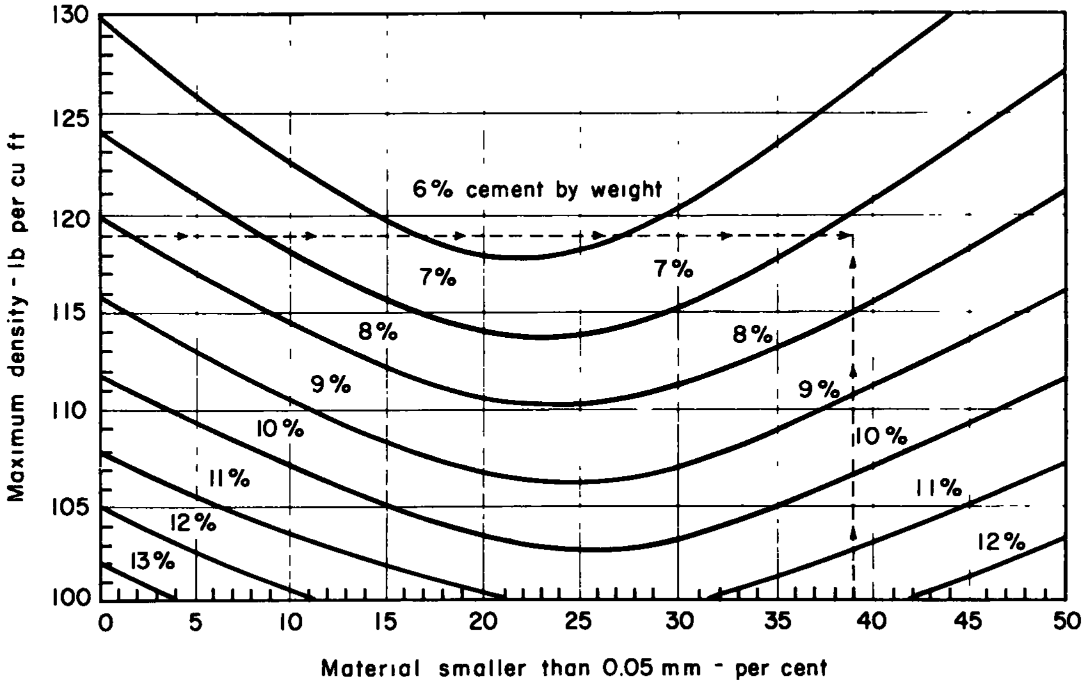

Figure 2. Indicated cement content of soil-cement mixtures having no material retained on the No. 4 sieve.

strength specimens[2] in triplicate at maximum density and optimum moisture content.

Step 4 : Determine the average compressive strength of the specimens after 7 days moist curing.

Step 5 : On Figure 3, plot the average compressive strength value obtained in Step 4. If this value plots above the curve, the indicated cement factor by weight, determined in Step 2, is adequate. For field construction use Figure 4, to convert this cement content by weight to a volume basis.

Note 2 : If the average compressive strength value plots below the curve of Figure 3, the indicated cement factor obtained in Step 2 is probably too low. Additional tests will be needed to establish a cement requirement. These tests generally require the molding of two test specimens, one at the indicated cement factor obtained in Step 2 and one at a cement content two percentage points higher. The specimens are then tested by ASTM-AASHO freeze-thaw test procedures.

Method B

Step 1 : Determine by test the maximum density and optimum moisture content for a mixture of the soil and portland cement.[3]

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[2] Specimens of 2-in. diameter by 2-in. height or 4-in. diameter by 4.6-in. height may be molded. The 2-in. specimens shall be submerged in water one hour before testing and the 4-in. specimens four hours. The 4-in. specimens shall be capped before testing.

[3] "Method of Tests for Moisture-Density Relations of Soil-Cement Mixtures," ASTM Designation D558.

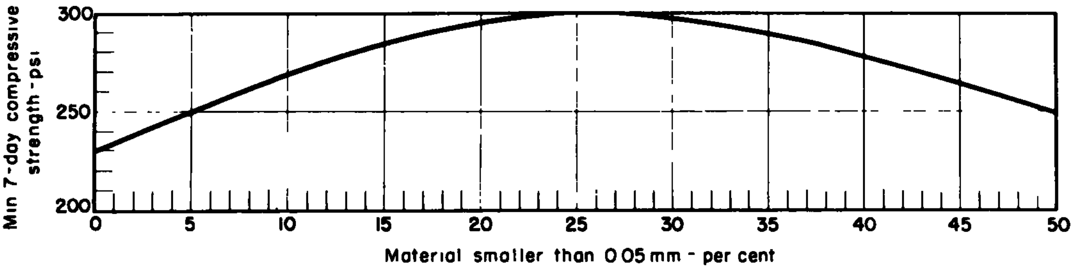

Figure 3. Minimum 7-day compressive strengths required for soil-cement mixtures having no material retained on the No. 4 sieve.

Note 3 : Use Figure 5 to determine an estimated maximum density of the soil cement mixture being tested. This estimated maximum density, the percentage of material smaller than 0.05 mm (No. 270 sieve) and the percentage of material retained on the No. 4 sieve may be used with Figure 6 to determine the cement content by weight to use in the test.

The soil sample for the test shall contain the same percentage of material retained on the No. 4 sieve as the original soil sample contains. However, ¾-in. material is the maximum size used. Should there be material larger than this in the original soil sample, it is replaced in the test sample by an equivalent weight of material passing the ¾-in. sieve and retained on the No. 4 sieve.

Step 2 : Use the maximum density obtained by test in Step 1 to determine from Figure 6 the indicated cement requirement.

Step 3 : Use total material as described in Step 1 and the indicated cement factor obtained in Step 2 to mold compressive strength specimens[4] in triplicate at maximum density and optimum moisture content.

Step 4 : Determine the average compressive strength of the specimens after 7 days moist curing.

Step 5 : Determine from Figure 7 the minimum allowable compressive strength for the soil-cement mixture. If the average compressive strength obtained in Step 4 equals or exceeds the minimum allowable strength, the indicated cement factor by weight obtained in Step 2 is adequate. For field construction, use Fig-

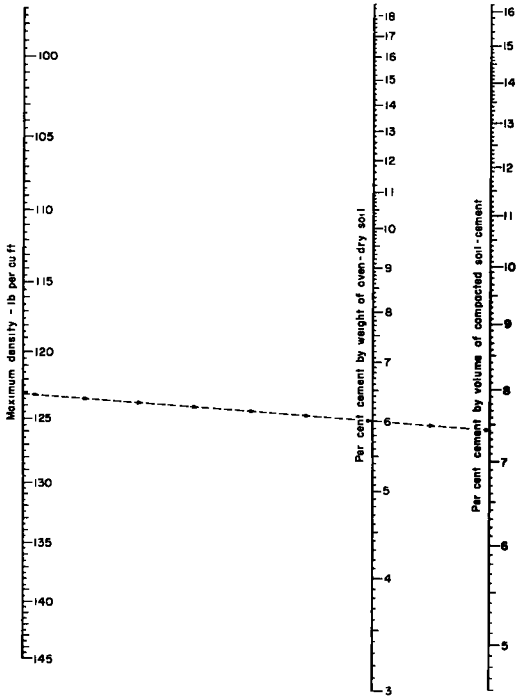

Figure 4. Relation of cement content by weight of oven-dry soil to cement content by volume of compacted soil-cement mixture.

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[4] Specimens of 4-in. diameter by 4.6-in. height shall be molded. They shall be submerged in water four hours and shall be capped before testing.

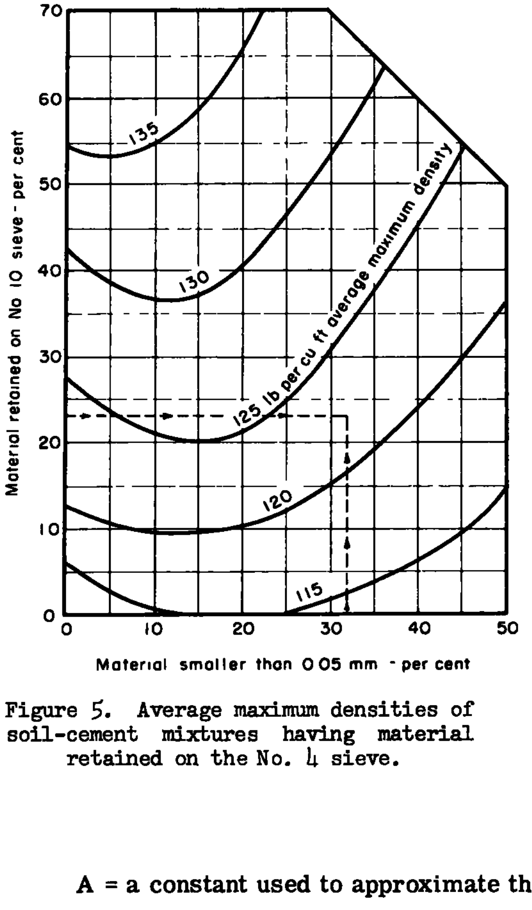

Figure 5. Average maximum densities of soil-cement mixtures having material retained on the No. 4 sieve.

ure 4 to convert this cement content by weight to a volume basis.

Note 4: If the average compressive strength value is lower than the minimum allowable, the indicated cement factor obtained in Step 2 is probably too low. Additional tests as described in Note 2 are needed.

DEVELOPMENT OF CHARTS AND PROCEDURE FOR METHOD B

The original charts developed in 1952 (Method A) were based on the fraction of the soil-cement mixture passing the No. 4 sieve. To develop Method B, these charts were converted to factors expressed in terms of total sample. These conversions are described below.

Chart for Determining Indicated Cement Content

Figure 6, used to determine the indicated cement content of soil-cement mixtures containing material retained on the No. 4 sieve, was developed from Figure 2. First, the original curves in Figure 2 were approximated by a family of parabolas representing cement contents.

These parabolas are represented by the equation:

$$c = \frac{A\ (s-24)^2 - d + 140.4}{4} \qquad (1)$$

where:

c = cement requirement of the fraction passing the No. 4 sieve, percent by weight.

A = a constant used to approximate the curves in Figure 2.

s = percent material smaller than 0.05 mm in the fraction of the soil passing the No. 4 sieve.

d = maximum density of soil-cement mixture passing the No. 4 sieve, lb per cu ft.

Next, the above equation was expressed in terms of the total sample. To do this it was necessary to determine the relationship between each variable expressed in terms of the fraction passing the No. 4 sieve and the corresponding variable when expressed in terms of the total sample. These relationships can be expressed by the following equations:

$$d = \frac{D-0.9\ RG}{(1-R)} \qquad (2)$$

$$c = \frac{C}{(1-R)} \qquad (3)$$

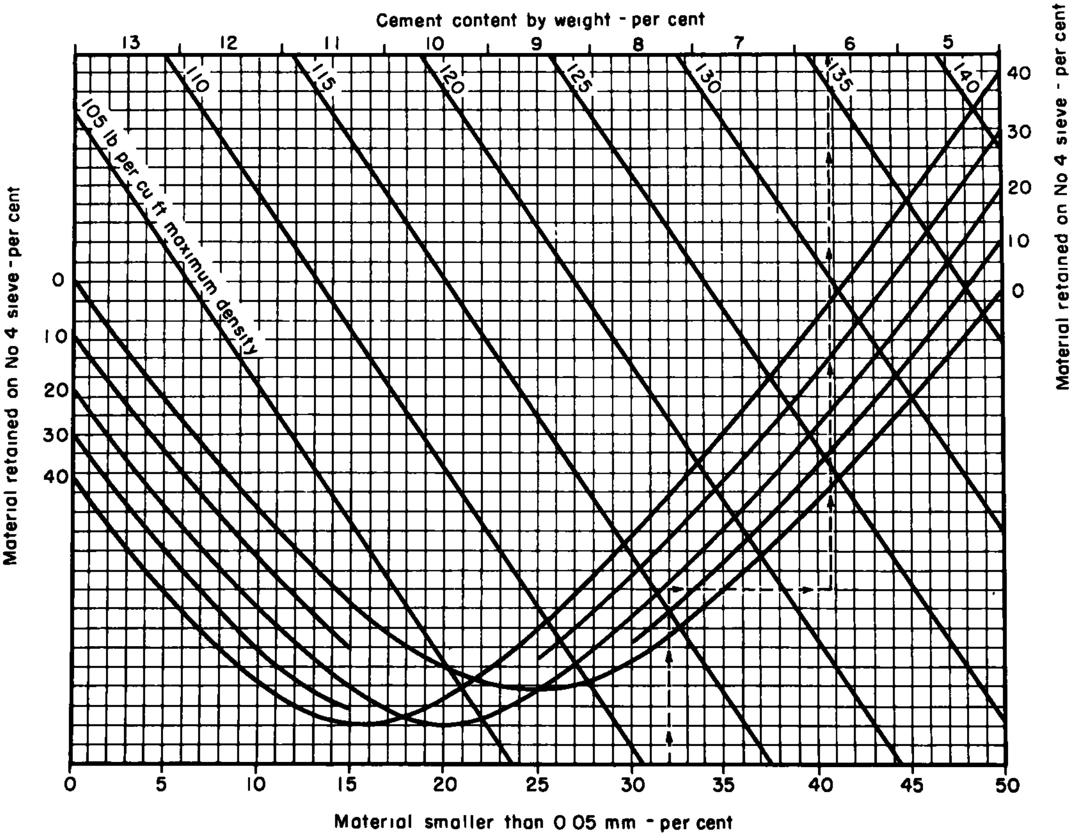

Figure 6. Indicated cement content of soil-cement mixtures having material retained on the No. 4 sieve.

$$s = \frac{S}{(1-R)} \tag{4}$$

Where: d = maximum density of soil-cement mixture passing the No. 4 sieve, lb per cu ft.

D = maximum density of total soil-cement mixture, lb per cu ft.

R = percent of material retained on the No. 4 sieve divided by 100.

G = bulk specific gravity of material retained on the No. 4 sieve x 62.4.

c = cement requirement of the fraction passing the No. 4 sieve, percent by weight.

C = cement requirement of total sample, percent by weight.

s = percent material smaller than 0.05 mm in the fraction of soil passing the No. 4 sieve.

S = percent material smaller than 0.05 mm in the total sample.

Equations 2, 3 and 4 were then substituted in Equation 1. This resulted in the following equation used to construct Figure 6:

$$C = \frac{(1-R)\,A\left(\frac{S}{1-R} - 24\right)^2 - D + 140.4 + 0.9RG - 140.4R}{4} \qquad (5)$$

To simplify the construction and use of Figure 6, an average value of 156.0 (250 x 62.4) was used for G in Equation 5.

The validity of using Equations 2, 3 and 4 for constructing Figure 6, as described above, is based on a study of the following relationships:

Density. It had been the practice in soil-cement testing to determine the maximum density on the fraction passing the No. 4 sieve and then to calculate the theoretical maximum density of the total mixture. These calculations assumed that the addition of material retained on the No. 4 sieve increases the density of the soil-cement mixture by displacing, in equal volume, the mixture passing the No. 4 sieve. No allowance was made for possible increase in void space. As a result, the densities obtained in test specimens were often considerably less than the calculated maximum density. The difference was in proportion to the amount of material retained on the No. 4 sieve. In some cases, actual densities of test specimens containing about 40 percent material retained on the No. 4 sieve would be as much as 10 lb per cu ft below the calculated theoretical density. This difference apparently was due to the voids not accounted for in the calculations.

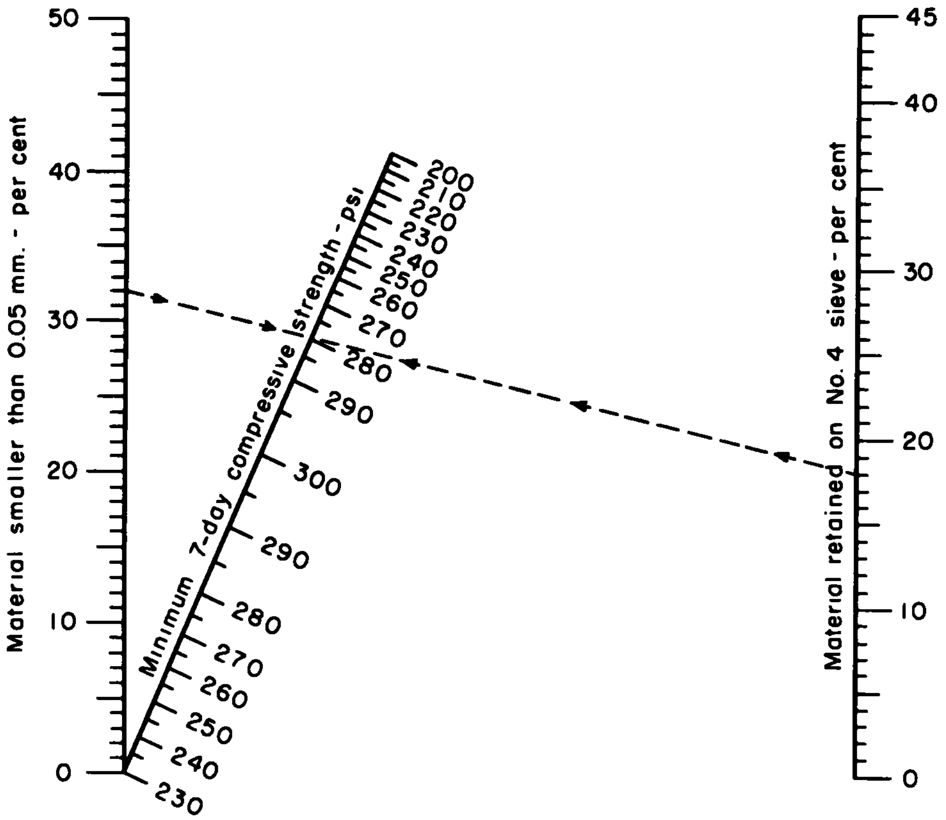

Figure 7. Minimum 7-day compressive strengths required for soil-cement mixtures having material retained on the No. 4 sieve.

The Civil Aeronautics Administration (6) has used a correction equation for determining the relationship between the maximum density of the total sample and the max-

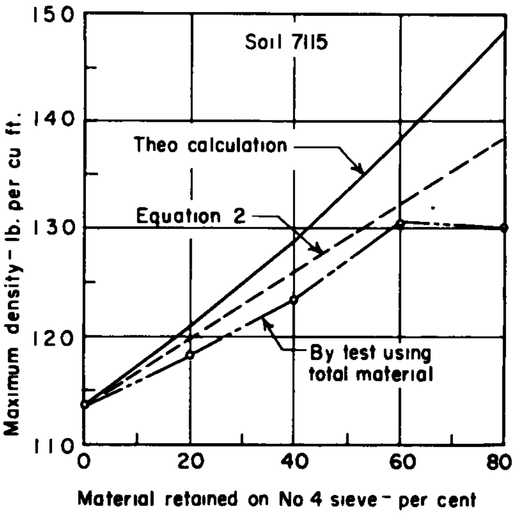

Figure 8. Effect of material retained on the No. 4 sieve on the maximum density, Soil No. 7115.

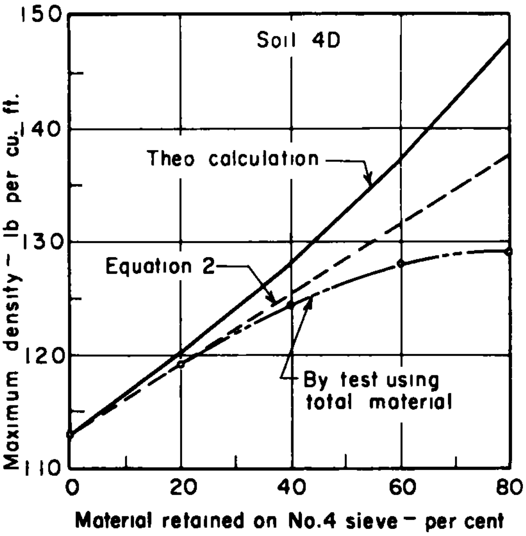

Figure 9. Effect of material retained on the No. 4 sieve on the maximum density, Soil No. 4D.

imum density of the fraction passing the No. 4 sieve. Equation 2 expresses this relationship.

Mainfort and Lawton (7) reported further studies on compaction tests using total material. They concluded that the correction formula may be used to predict maximum densities obtained on the total material if the amount of material retained on the No. 4 sieve is not over 40 to 60 percent. Most soil-cement mixtures contain less than this. Specifications for soil-cement generally limit the amount to 45 percent.

The Portland Cement Association made similar studies by comparing the maximum densities obtained by test on the total material and maximum densities obtained by Equation 2. Typical results are given in Figures 8 and 9. Also given is a comparison of the maximum densities calculated on the assumption that the addition of material retained on the No. 4 sieve increases the density of the mixture by displacing, in equal volume, the mixture passing the No. 4 sieve.

Soil No. 7115 (Fig. 8) is a natural gravel and sand. Material retained on the No. 4 sieve was separated from the material passing the No. 4 sieve. It was then recombined in the desired proportions. Soil No. 4D (Fig. 9) is an A-4 silt loam to which were added varying percentages of material retained on the No. 4 sieve.

Thus, if the amount of material retained on the No. 4 sieve is not over 40 to 60 percent the maximum densities obtained with Equation 2 approximate quite closely the maximum densities obtained by test using total material. This further shows the validity of using Equation 2 to convert the short-cut charts to total material.

Cement Content. The original short-cut test method indicated a cement content adquate for the fraction of the soil-cement mixture passing the No. 4 sieve. This was then converted by charts to a cement requirement of the total sample. The validity of this conversion was determined in the original 1952 correlation (1). This showed that the cement requirement of the fraction passing the No. 4 sieve is unchanged when material retained on the No. 4 sieve is added. Because the material retained on the No. 4 sieve requires no extra cement, the requirement of the total sample lessens proportionally when that material is added. Thus, Equation 3 which expresses this relationship can be used to convert the original short-cut charts to total material.

Material Smaller than 0.05 mm . It was next necessary to consider the relationship between the amount of material smaller than 0.05 mm in the fraction of the soil

TABLE 1

COMPARISON OF COMPRESSIVE STRENGTHS OF SOIL-CEMENT SPECIMENS

	7-Day Compressive Strength			
	2-in. diameter x 2-in high		4-in. diameter x 4.6-in. high	
Soil No.[1]	Number of Specimens	Average Strength, psi	Number of Specimens	Average Strength, psi
6006	3	458	3	467
6017	3	276	3	282
5992	3	564	3	575
4d	3	307	3	337
6018	3	283	3	261
6023	3	385	3	362
6046	3	388	3	421
S-1	3	239	2	192
TD-2	3	327	2	335
6741	3	356	2	341
6739	3	671	2	624
6738	3	420	2	417
6718	3	282	2	280

[1] The fraction of the soil passing a No. 4 sieve only used in test specimens.

sample passing the No. 4 sieve and the amount smaller than 0.05 mm in the total sample. This relationship is, by proportion, that expressed in Equation 4.

Chart for Determining Minimum 7-Day Compressive Strengths

Figure 7 gives the minimum allowable 7-day compressive strengths for soil-cement mixtures having material retained on the No. 4 sieve. The chart was developed from Figure 3. Thus, the minimum compressive strength criteria for Method A (Fig. 3) is the basis for Figure 7 used with Method B. The validity of using the Method A minimum compressive strength criteria for Method B is based on the following studies:

1. To determine how the compressive strengths are affected by adding material retained on the No. 4 sieve, holding constant the cement content by weight in the fraction passing the No. 4 sieve.
2. The effect of using 4-in. diameter by 4.6-in. high compressive strength specimens required for Method B rather than 2-in. diameter by 2-in. high specimens which may be used for Method A.

Results of these studies are given in Figure 19 and Table 1. They indicate that the compressive strength is not changed appreciably by the two factors. Thus, data from specimens of either size may be used interchangeably. Further, the minimum compressive strength criteria given in Figure 3 based on the fraction passing the no. 4 sieve, may be used with the new short-cut procedure for soils containing material retained on the No. 4 sieve (Method B). The use of Figure 3 with Method B, however, would make it necessary to calculate, for each sandy soil being tested, the quantity of material smaller than 0.05 mm in the fraction passing the No. 4 sieve. To avoid this calculation, Figure 7, in which the quantity of material smaller than 0.05 mm in the total sample is used directly, was developed. To do this, a nomograph based on Equation 4 was constructed. Then the original plotted values of "s" on the diagonal line were replaced with the corresponding values of compressive strength obtained from Figure 3.

Chart for Determining Average Maximum Densities

Method A uses Figure 1 for estimating maximum densities of soil-cement mixtures having no material retained on the No. 4 sieve. However, Figure 1 is not accurate for Method B using soils containing material retained on the No. 4 sieve. Therefore, more accurate relationships for determining approximate maximum densities for Method B were determined by plotting a number of gradation or "soil separate" factors agains the actual maximum densities obtained by tests on 209 soils. Two factors—the percent material smaller than 0.05 mm and the percent material retained on the No. 10 sieve—gave the best approximation. They are the basis for Figure 5.

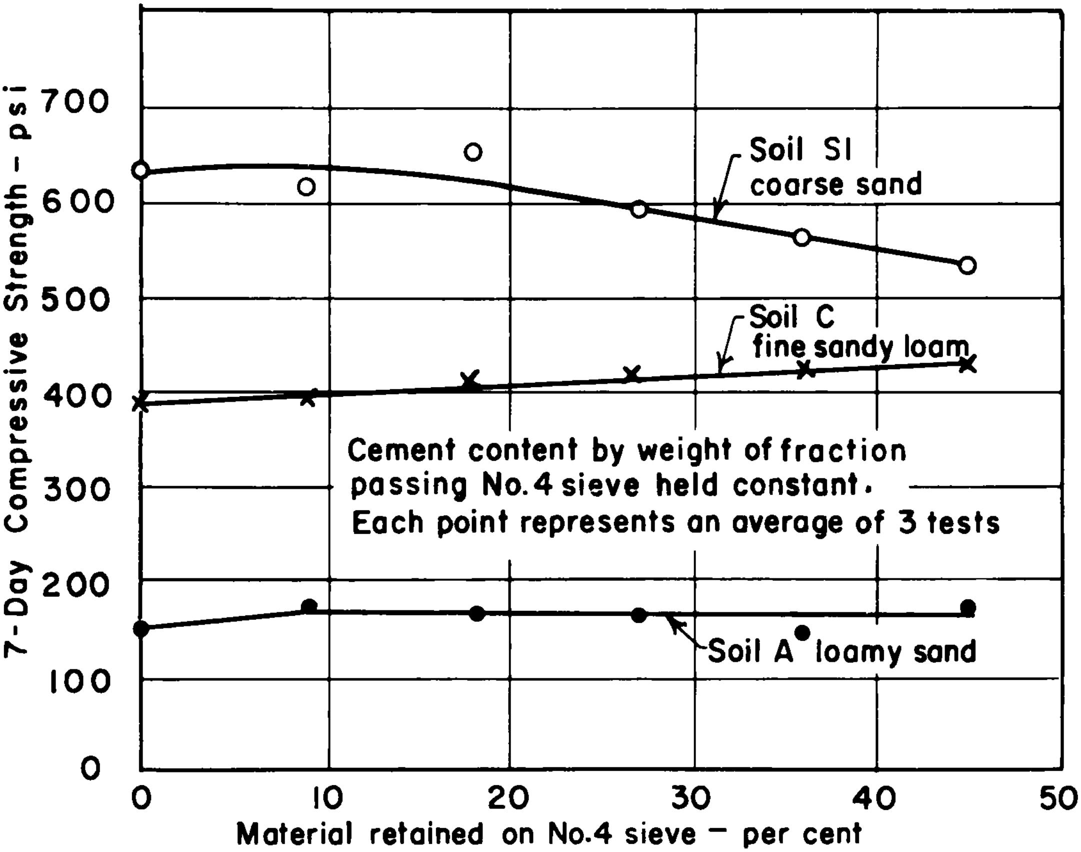

Figure 10. Effect of material retained on the No. 4 sieve on compressive strength of soil-cement specimens.

RESULTS OBTAINED IN CHECKING TEST PROCEDURE FOR METHOD B

The Portland Cement Association tested the accuracy of Method B by comparing the cement requirements obtained by this method and the requirements obtained on total material tested by ASTM wet-dry and freeze-thaw methods (4, 5). The comparison considered results of tests of 209 soils. First, a cement factor for each soil was selected from Figure 6. This factor was considered adequate if the compressive strength at this cement content equaled or exceeded the minimum allowable strength given in Figure 7. If the strength were lower than the minimum allowable, additional testing was considered necessary to determine the cement requirement. The cement factors thus obtained were then compared with those obtained from ASTM tests. The comparison showed the following results:

Adequate cement factors, or the need for further testing, were indicated for 204 (97.6 percent) of the 209 soils. For 5 of the soils (2.4 percent) the procedure did not indicate adequate cement factors nor did it indicate the need for further testing. The strengths for these soils at the indicated cement content were higher than the minimum allowable strengths given in Figure 7.

Using results obtained from ASTM wet-dry and freeze-thaw tests as a yardstick, the accuracy of Method B is shown in Figure 11. The reliability of 97.6 percent compares favorably with the 98.6 percent reported in the original 1952 correlation (1).

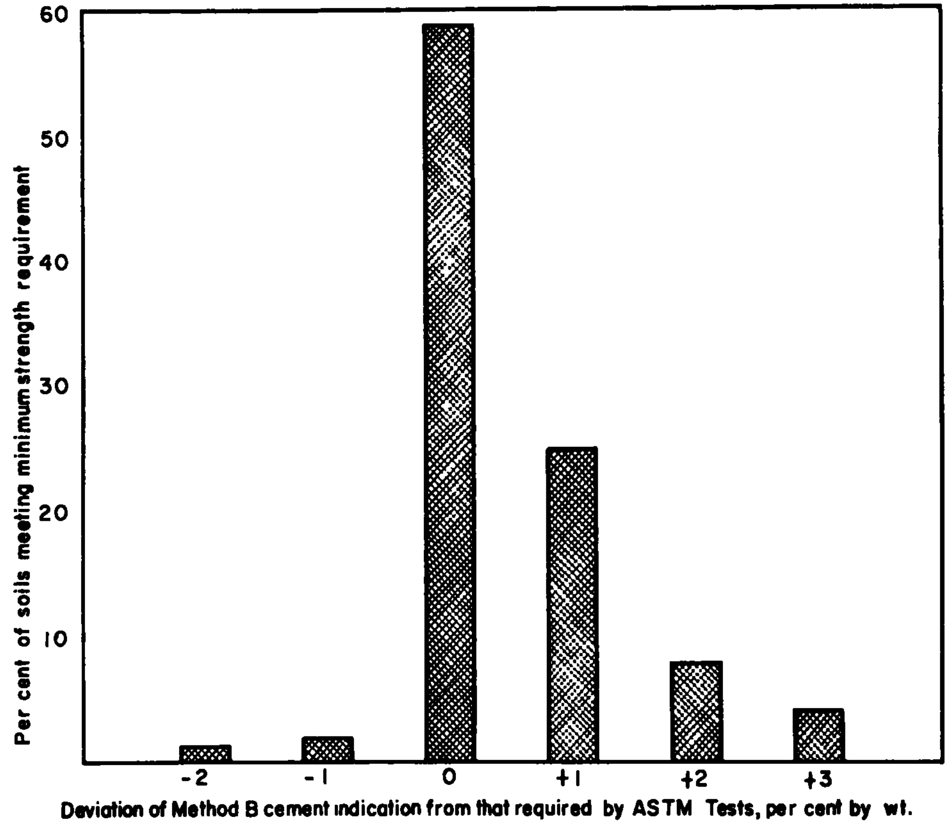

Figure 11. Accuracy of short-cut test (Method B).

SUMMARY

In 1952, the Portland Cement Association developed a short-cut procedure for determining cement requirements for sandy soils. The short-cut procedure was based on a correlation of data from ASTM or AASHO tests on 2,229 sandy soils. The method permits all tests to be completed in one day, except the 7-day compressive strength tests.

The original charts for the short-cut procedure were based on the fraction of the soil-cement mixture passing a No. 4 sieve. For soils containing material retained on the No. 4 sieve it was necessary first to determine the cement requirement of the fraction passing the No. 4 sieve and then to convert this to the cement requirement of the total mixture. Similarly, the maximum density and optimum moisture content of the total soil-cement mixture were calculated for field construction based on tests on the fraction passing the No. 4 sieve.

Soil-cement testing of soils containing material retained on the No. 4 sieve now requires the use of total material. Therefore, for soils with material retained on the No. 4 sieve, the short-cut charts were converted from relationships based on the fraction passing a No. 4 sieve to relationships expressed in terms of the total sample.

As a result, two procedures are given for determining cement factors: Method A for soils having no material retained on the No. 4 sieve and Method B for soils containing material retained on the No. 4 sieve. Method A uses charts developed in 1952. Method B, newly developed, uses charts based on the total material.

The reliability of Method B was checked with ASTM-AASHO test data for 209 sandy soils. The procedure provided safe cement factors or showed the need for further testing for 97.6 percent of the soils. While the cement factors obtained were not always the minimum that could be used, they were practical.

The procedure and chart should be used in their present form until local test data and experience permit revision of the charts for local soil conditions.

REFERENCES

1. Leadabrand, J. A. and Norling, L. T., "Soil-Cement Test Data Correlation Affords Method of Quickly Determining Cement Facors for Sandy Soils." Highway Research Board Bulletin 69 (1953).
2. Leadabrand, J. A. and Norling, L. T., "Simplified Methods of Testing Soil-Cement Mixtures." Highway Research Board Bulletin 122 (1956).
3. Method of Test for Moisture-Density Relations of Soil-Cement Mixtures; ASTM Designation D558-57.
4. "Method of Wetting-and -Drying Test of Compacted Soil-Cement Mixtures." ASTM Designation D559-57.
5. "Method of Freezing-and-Thawing Test of Compacted Soil-Cement Mixtures." ASTM Designation D560-57.
6. "Standard Specifications for Construction of Airports." U.S. Department of Commerce, CAA Office of Airports, Washington, D.C., p. 572 (January 1948).
7. Mainfort, R. C. and Lawton, W. L., "Laboratory Compaction Tests of Coarse-Graded Paving and Embankment Materials." Proceedings, HRB (January 1953).

A Rapid Method of Predicting the Portland Cement Requirement for Stabilization of Plastic Soils

SIDNEY DIAMOND and EARL B. KINTER, Highway Physical Research Engineers,
Division of Physical Research, Bureau of Public Roads

The relation between the cement requirement of soil-cement mixtures and surface area determined by the glycerol retention method was investigated for a group of plastic soils. For soils containing less than 45 percent silt, a strong correlation was found between surface area and the cement content at which 10 percent loss occurred during 12 cycles of the freeze-thaw durability test. A regression equation and procedure were developed for predicting cement requirement from surface area. In a comparison of predicted cement requirements with those derived from freeze-thaw test data, an average deviation of 0.6 percent cement was obtained.

● THE USE OF SOIL-CEMENT for highway base courses has expanded rapidly in recent years, especially in areas where local sources of granular deposits are nonexistent or rapidly being depleted. Present methods for determining the amount of cement necessary to produce a stable material from a given soil involve the use of rather large soil samples and an extended testing program. The freeze-thaw or wet-dry durability tests (1) in particular require considerable time and effort. A short-cut method developed by the Portland Cement Association (P.C.A.) provides an easier alternate procedure for sandy soils, but extended testing is still required for plastic materials. This report is concerned primarily with these plastic materials, and with the development of a shorter, more economical test procedure for estimating their cement requirements.

Despite a number of years of successful use of soil cement, little is known of the physical chemistry of the reactions between the soil and the cement. It has been noted that the more plastic soils generally require larger quantities of cement for effective stabilization. Although there does not seem to be any consistent relation between cement requirement and plasticity index or any of the familiar engineering tests, it was thought that perhaps such a relationship might exist with the surface area of the soil.

Some years ago, Catton, exploring this proposed relation by the use of surface area values computed from grain-size distribution curves, found an almost complete lack of correlation (2). However, an examination of Catton's data reveals that the largest values he computed for surface area were just over 2 square meters per gram (m^2/g), even for A-6 and A-7 soils. Present knowledge indicates that actual surface areas are many times higher than this, often as high as several hundred m^2/g. Catton's low values of surface area could not, therefore, be expected to provide any real correlation.

The method widely considered as "standard" for realistically determining the surface area of fine-grained materials, is known as the Brunauer-Emmet-Teller (B.E.T.) Method, and involves the adsorption of nitrogen or some other gas on the surface of the material at low temperatures. The complicated apparatus and time-consuming procedures required are, however, not suitable for general routine determinations. Further, the method is not particularly applicable to soils since the non-polar gases normally used do not measure the internal surfaces of expanding clay minerals which are present in many soils. The crystals of the clay minerals are formed in sheet-like layers; in an expanding mineral the flat surfaces of the inner layers constitute the internal surface.

TABLE 1

DESCRIPTION OF SOILS

Identification						Physical Properties - Data Furnished by P C A													
						Gradation - percent finer than													
						Sieve number													
B P R soil No	P C A soil No	State	County	Soil series	Soil horizon	4	10	40	200	0 05 mm	0 005 mm	0 002 mm	Liquid limit	Plasticity index	AASHO classification	Optimum moisture of soil-cement mixture(%)	Maximum density of soil-cement mixture(pcf)	Cement requirement (P C A) % by vol	Surface area of whole soil m²/g
A (First group)																			
S-32044	7538	Ala.	Madison	-	-	100	85	61	24	21	6	5	17	4	A-2-4(0)	11 4	115 5	7 0	12 5
S-32045	7494	Ala	Montgomery	-	B	100	100	98	70	48	29	28	35	11	A-6(7)	17 7	104 0	13 5	90
S-32046	7537	Ala.	Montgomery	-	B	100	100	94	43	38	29	27	34	12	A-6(2)	17 9	108.0	10 5	49.5
S-32047	7509	Ark	Ashley	Richland	A B C	100	100	98	96	92	27	20	32	11	A-6(9)	16 2	105.5	9 5	63
S-32050	7489	Ark	Ashley	Portland	A	100	99	95	92	90	23	15	27	6	A-4(8)	17 3	103 5	17±	59
S-32051	7490	Ark	Ashley	Portland	B	100	100	97	94	90	23	18	30	9	A-4(8)	18 2	104 7	11 0	79
S-32053	7552	Ark	Nevada	Ruston	C -	100	100	70	30	28	22	18	39	17	A-2-6(1)	15 0	109.0	7.5	48
S-32055	7555	Ark.	Nevada	Ruston	A. B. C	100	99	99	38	32	22	18	20	6	A-2-4(0)	12.7	119 0	7 0	37 5
S-32056	7542	Ark	White	-	A	100	99	97	76	56	19	12	22	5	A-4(8)	12.3	112.6	11.0	36
S-32060	7375	Idaho	Idaho	-	-	55	27	15	9	9	2	2	26	5	A-1-a(0)	10.0	134.7	9 0	14
S-32061	7497	Ill.	Cook	-	-	55	47	35	19	15	7	5	25	6	A-1-b(0)	8 0	125 3	7.0	12
S-32062	7498	Ill.	Cook	-	-	100	99	97	91	88	42	28	41	16	A-7-6(11)	23 4	96 7	14 5	81
S-32063	7h	Ill	Cook	-	-	100	100	97	90	87	48	-	41	24	A-7-6(14)	17 5	104 5	13 0	77 5
S-32064	7460	Ill	Henry	-	-	59	46	28	16	16	8	6	26	6	A-1-b(0)	9 4	130 3	7 0	7 5
S-32065	7558	Ill	Iroquois	Hagner	-	85	83	78	33	22	9	3	18	7	A-2-4(0)	10 2	121 0	7 5	25 5
S-32066	7528	Ill	Massac	-	-	80	74	67	56	49	10	7	22	1	A-4(4)	11 3	117 6	8 5	26
S-32067	7529	Ill	Massac	-	-	77	71	66	60	58	16	13	26	6	A-4(5)	13 3	115 8	10.5	40.5
S-32068	7530	Ill.	Massac	-	-	75	63	50	35	32	9	8	19	4	A-2-4(0)	11 0	120 8	9 0	28 5
S-32069	7560	Ill.	Massac	-	-	55	48	41	36	34	11	8	30	9	A-4(0)	11 2	119 5	7 0	22.5
S-32070	7561	Ill.	Massac	-	-	84	76	73	39	23	8	8	20	3	A-4(1)	11.7	118 6	7.0	19.5
S-32071	7562	Ill.	Massac	-	-	80	71	66	56	51	15	12	26	5	A-4(4)	13.0	117.4	9.5	37.5
S-32072	7563	Ill.	Massac	-	-	55	47	34	12	11	8	8	28	15	A-2-6(0)	9 4	126 8	7 0	14
S-32074	S-7	Ill	McHenry	-	B. C	100	99	99	92	79	13	-	26	7	A-4(8)	15 0	113.0	9.0	42.5
S-32075	6900	Kan.	Grant	-	-	100	97	94	77	69	25	19	29	9	A-4(8)	16 5	106 7	10.5	87 5
S-32076	7520	Ky	Carter	-	-	59	56	52	24	13	4	3	19	3	A-2-4(0)	7 8	130 2	6 5	6
S-32078	7515	Ind	Vandenb'g	-	-	100	100	94	27	26	15	12	20	4	A-2-4(0)	12 2	120 0	7.5	20
S-32082	7525	Mo	Jackson	-	C	100	100	100	94	79	17	14	27	5	A-4(8)	15 5	108 7	9 0	82
S-32083	7526	Mo.	Jackson	-	-	100	100	100	94	80	15	13	28	6	A-4(8)	15 7	108.0	9 5	85 5
S-32085	7514	Okla.	Okmulgee	-	A	92	86	85	36	25	15	10	27	7	A-4(0)	12 0	117 6	7 0	32 5
B (Second group)																			
S-32572	7682	Colo.	Fremont	-	-	67	52	39	29	27	16	10	26	9	A-2-4(0)	11 8	124 2	11 0	14 5
S-32573	7687	Ill.	Will	-	B	100	100	99	97	93	45	37	47	25	A-7-6(15)	22 0	100 3	15.0	143
S-32575	7695	La	Livingston	-	-	100	99	98	94	86	17	12	26	4	A-4(8)	17 7	104 4	15 5	33
S-32576	7701	Tenn	Franklin	-	B	100	99	88	53	48	32	26	32	15	A-6(6)	16 9	109 4	10 0	36
S-32577	7702	Tenn	Franklin	-	A	98	96	83	40	36	21	17	25	10	A-4(1)	13 7	113 5	7 0	34
S-32578	7761	Mont	Silver	-	-	96	83	46	22	20	10	9	30	9	A-2-4(0)	11 8	119 5	7 5	34
S-32579	7762	Mich.	Calhoun	-	A B.	92	87	75	37	31	13	9	23	8	A-4(0)	7 9	115 9	10.0	37
S-32582	7776	Kan	Douglas	-	-	100	100	100	97	85	16	14	29	5	A-4(8)	21 0	97 0	12 0	78
S-32583	7777	Kan.	Douglas	-	-	100	100	100	98	86	16	14	29	6	A-4(8)	18 5	102 2	12 0	73 5
S-32584	7779	Tex	Wichita	-	-	55	39	25	16	13	5	5	25	9	A-2-4(0)	8 4	129 0	7 0	18

Lately, a method (3) involving the retention of ethylene glycol by soils has also been used to estimate surface area, but there is some doubt as to the specific quantitative relation between the actual surface area and the amount of ethylene glycol retained under the conditions of the determination. More recently, a new and simpler method involving the retention of glycerol, has been developed at the laboratories of the Bureau of Public Roads (4, 5).

Values of surface area obtained for various clays by this method were shown to agree closely with theoretical values of expanding clays and, for non expanding clays, with those determined by the B. E. T. Method. In the present work, surface areas were measured by this method on a number of soils of known cement requirement. The correlation between the resulting surface area values and cement requirements was then determined. An equation was derived for predicting cement requirements from surface area values, and the predicted cement requirements were compared with the actual cement requirements as determined by test.

MATERIALS AND METHODS

Soils

Soil samples and accompanying engineering test data were supplied by the Portland Cement Association.[1] Of the soils furnished, only those with measurable plasticity indexes were included in this study. The soils of the first group received are described in part A of Table 1. They were not selected as representative of any particular soil area or type of soil but were simply those on hand at the P. C. A. laboratory at the time this study was initiated. They do, however, include samples from a number of different states and of various soil horizons. The last ten soils listed in Table 1

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[1] The cooperation of the Portland Cement Association in supplying the soil samples and accompanying test data is gratefully acknowledged. Thanks are especially due to Mr. J. A. Leadabrand and Mr. L. T. Norling, of the Soil Cement Bureau for their interest.

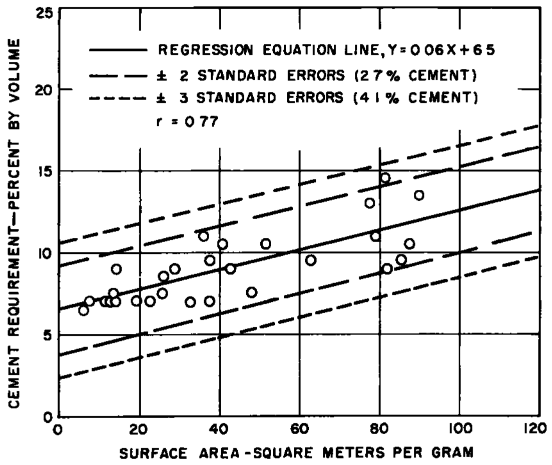

Figure 1. Recommended cement requirement (P.C.A., percent by volume) vs surface area of soils (first group).

(part B) were received as a second group and represent samples processed by the P.C.A. laboratory during the period in which this study was made.

Surface Area Measurements

In the glycerol retention method advantage is taken of the ability of clay and other soil constituents to adsorb glycerol molecules on their surfaces. Conditions are maintained under which only a single layer of the glycerol molecules is adsorbed and retained. The amount of glycerol adsorbed is measured by weighing the sample before and after treatment, and the weight of the adsorbed glycerol can be related to the surface area of the sample.

As applied to this study, the method involves the following steps:

1. Dry duplicate small samples (about 1 gm each) of the passing 40-mesh fraction of the soil at 110 C in aluminum foil dishes, and weigh to 0.0002 gm on an analytical balance.
2. Add 10 ml of a dilute (2 percent) water solution of glycerol to the sample, and swirl the container gently to mix the contents.
3. Heat at 110 C (±3 C) in a mechanical-convection oven containing a supply of glycerol to provide a source of free glycerol vapor in the oven chamber. Under these conditions, glycerol in excess of a monomolecular layer and water are both removed.
4. Reweigh after equilibrium has been attained, normally after over-night heating. The gain in weight over the original oven-dry weight of the sample is due to the monomolecular layer of glycerol adsorbed on both the internal and external surfaces. The adorbed glycerol is expressed as a percentage of the 110 C dry weight of the soil.
5. A distinction must be made between that portion of the glycerol retained on external surfaces of all clay minerals and that retained on internal surfaces of expanding minerals such as montmorillonite and vermiculite. On the internal surfaces both the top and bottom of the monomolecular layer of glycerol are in contact with clay surfaces. On the outside of the particles, however, only one side of the monomolecular layer is in contact with clay surface. Therefore, a given amount of glycerol on internal surfaces accounts for twice as much clay surface area as the same amount would if it were on external surfaces.

To make this distinction, a second determination is required. This is accomplished by determining the percentage of glycerol retained by replicate samples previously heated to 600 C, the glycerol retention being determined by the same procedure as described above. Heating to 600 C normally collapses and irreversibly closes the internal spaces and thus renders them inaccessible to glycerol molecules. The difference between the original percentage of glycerol retained and that retained after heating to 600 C is attributable to internal surfaces; the percentage measured after this preliminary heating is due to external surfaces only

Based on x-ray diffraction evidence concerning the thickness of a monomolecular layer of glycerol, it has been shown that one-hundredth of a gram of glycerol covers 35.3 sq m of internal clay surfaces (5); thus a glycerol retention of 1 percent on internal surfaces corresponds to 35.3 m^2/g. Similar deductions indicate that a retention of 1 percent glycerol on external surfaces corresponds to a specific surface of 17.65 m^2/g.

For the soils used in this study the surface area value of the whole soil was computed by multiplying the surface area found for the passing 40-mesh fraction by the percentage of the whole soil which passes the 40-mesh sieve. The surface area of the particles coarser than 40 mesh is so small as to be negligible. A hypothetical example of these computations is as follows:

Glycerol retention of passing 40-mesh fraction:	3.50 percent
Glycerol retention of same after preliminary 600 C heating:	1.50 percent
Retention due to external surface:	1.50 percent
Retention due to internal surface: 3.50 - 1.50 percent	2.00 percent

Indicated surface area of passing 40-mesh fraction

External: $1.50 \times 17.65 = 26.5\ m^2/g$
Internal: $2.00 \times 35.3 = 70.6\ m^2/g$
Total $97.1\ m^2/g$

Percentage of whole soil passing 40-mesh sieve:	65 percent
Surface area of whole soil: $97.1\ m^2/g \times 0.65$	$63.1\ m^2/g$

For this study this figure would be rounded to the nearest half-square meter per gram, or 63 m^2/g.

Cement Requirements

The cement requirement determinations were performed by the staff of the P.C.A. Soil Cement Laboratory, using the methods described in their "Soil Cement Laboratory Handbook," 1956. Briefly, this method is as follows:

1. Determination of the grain-size distribution and Atterberg limits of the soil.
2. Determination of the moisture-density relations of a mixture of the soil and an assumed percentage of cement.
3. Molding durability test specimens at optimum moisture and at cement contents thought to bracket the cement requirement, and testing through 12 cycles of freezing and thawing. (Wet-dry tests may also be made, but were not used for the soils of this investigation.) For A-1, A-2-4, and A-2-5 soils, the cement requirement is specified by P.C.A. as that cement content at which test specimens lose 14 percent of their weight during the 12 cycles and the accompanying brushing procedure. For A-2-6, A-2-7, A-4, and A-5 soils, the loss permitted is 10 percent, and for A-6 and A-7 soils, it is 7 percent. These loss criteria are based on information from a great many laboratory tests, the performance of field projects, and outdoor exposure of several thousand specimens.
4. Checking the estimated cement factor by molding and testing small specimens for compressive strength to insure that adequate hardening takes place at this cement content.
5. For reporting and for field use, the cement factor is converted from a weight basis to a volume basis by use of the relation percent cement by vol= $\frac{D - \frac{D}{C}}{94} \times 100$, where D=oven-dry density of the soil-cement specimen in lb per cu ft, and C=100 plus the percent cement by weight of oven-dry soil, the quantity divided by 100.
6. The final recommended cement content is based to some extent on the judgment of the testing engineer. For example, the cement content indicated by the durability test data might be in a critical range, i.e., where a small decrease in cement content would lead to very much higher than allowable freeze-thaw losses. In such a case, inadequate mixing on the job could result in an unsatisfactory product, and to insure against this, the testing engineer would recommend a slightly higher over-all cement content than that provided for by the durability test data.

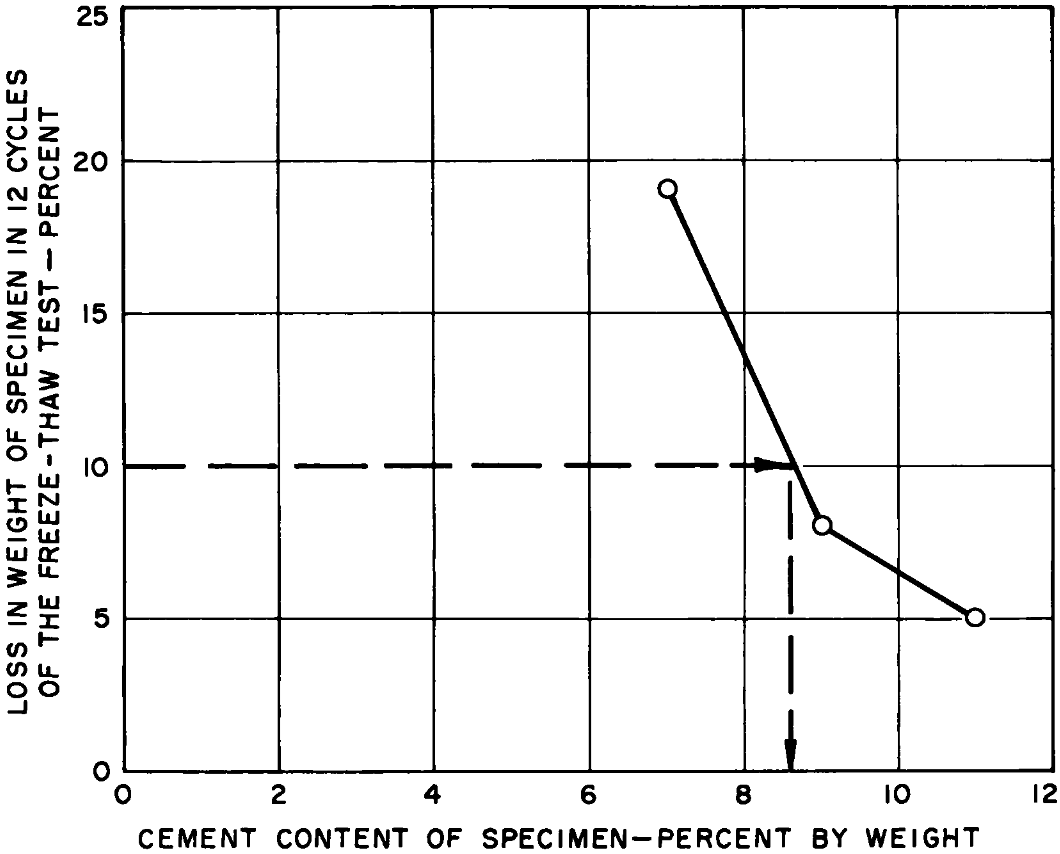

Figure 2. Method of determining cement content (by wt) at which 10 percent loss would occur in the freeze-thaw test (12 cycles). For this illustration the required value is 8.6 percent.

RESULTS AND DISCUSSION

Surface area values and cement requirements for the Group A samples (numbers S-32044 to S-32085) are listed in Table 1. The surface areas range from 6 to 90 m^2/g. The cement requirement values quoted (in terms of percent by volume) are those actually recommended for construction by the P. C. A. laboratory. Analysis shows that a statistically significant correlation[2] exists, $r = 0.77$, significant at the 0.1 percent level. The following regression equation was derived from the data: Cement requirement = 0.06 (surface area) + 6.5. The standard error of estimate from this equation is 1.37 percent cement by volume. A plot of the relation is given in Figure 1.

The statistical significance level of the correlation coefficient (0.1 percent) clearly indicates that a correlation actually exists. Nevertheless, the degree of correlation indicated by the correlation coefficient (0.77) is not strong enough to permit accurate predictions of cement requirement directly from surface area measurements.

Assuming the validity of the hypothesis that the surface area should be intimately associated with the cement need, a number of possible reasons for not obtaining the expected closer correlation could be deduced.

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[2] See appendix for definition of statistical terms.

1. The surface area values used are based on the weight of the soil, surface area being expressed in square meters per gram, whereas the cement requirements are expressed on a volume basis. This naturally would weaken the correlation because there can be no over-all relation between values expressed by weight and those expressed by volume, due to the variation in the densities of the different soil-cement products.

2. As noted earlier, the cement requirement determined by the P.C.A. laboratory is based on the cement content at which maximum losses of 7, 10, or 14 percent occur during the 12-cycle freeze-thaw test, the limit applying depending on the AASHO classification of the soil. Two soils containing different clay minerals might have identical surface areas, but have greatly different grain-size distributions and plasticity indexes, thereby falling into different classifications. Accordingly, the cement requirement of one might be based on the 7 percent loss limit and that of the other on the 14 percent loss limit, despite their identical surface areas. In effect, this inserts a bias in the cement requirement values which is not correspondingly reflected in the surface area, and so tends to weaken the correlation.

3. The cement requirements recommended are not derived solely from freeze-thaw loss data, but, as noted earlier, may be modified somewhat by engineering judgment. This added factor, while perfectly justified from a practical standpoint, is not related to a physical measurement such as surface area, and thus also tends to weaken the correlation.

4. The cement requirement data are rounded to the nearest half-percent of cement. Since the range in cement requirements encountered with these soils (6.5 to 14.5 percent) is only 16 times this figure, this rounding, while again justifiable from the practical standpoint, tends to weaken the correlation.

TABLE 2

TEST DATA FOR 12 CYCLES OF FREEZING AND THAWING AND DATA DERIVED FROM THEM

B.P.R. soil No.	Test data furnished by P.C.A.								Derived data			
	Specimen 1		Specimen 2		Specimen 3		Specimen 4		Cement content at 10% loss	Loss allowed freeze-thaw test	Cement content at which allowed loss occurs	
	Cement % by wt	Loss %	Cement % by wt	Loss %	Cement % by wt	Loss %	Cement % by wt	Loss %	% by wt	%	% by wt	% by vol
A (First group)												
S-32044	5	13	7	5	-	-	-	-	5.8	14	4.8	5.7
S-32045	10	13	12	10	14	6	-	-	12.0	7	13.5	13.2
S-32046	6	13	8	9	10	7	-	-	7.5	7	10.0	10.4
S-32047	8	10	10	5	12	4	-	-	8.0	7	9.2	9.7
S-32050	9	48	11	42	13	33	15	30	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32051	9	36	11	8	13	5	-	-	10.8	10	10.8	10.9
S-32053	7	23	9	2	-	-	-	-	8.2	10	8.2	8.8
S-32055	5	13	7	8	-	-	-	-	6.0	14	4.5[2]	5.4[2]
S-32056	8	35	10	7	12	6	14	5	9.6	10	9.6	10.5
S-32060	3	18	5	6	7	1	-	-	4.3	14	3.6	5.0
S-32061	5	3	7	2	9	2	-	-	n.a.[1]	14	n.a.[1]	n.a.[1]
S-32062	12	25	14	14	16	4	-	-	14.8	7	15.3	13.7
S-32063	8	13	10	10	12	8	-	-	10.0	7	12.9[2]	12.6[2]
S-32064	3	20	5	2	-	-	-	-	4.0	14	3.6	4.8
S-32065	4	100	6	11	8	7	-	-	6.5	14	5.9	7.0
S-32066	8	6	10	5	-	-	-	-	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32067	10	7	12	6	14	5	-	-	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32068	5	20	7	15	-	-	-	-	n.a.[1]	14	7.4[2]	8.7[2]
S-32069	6	7	8	5	-	-	-	-	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32070	4	18	6	9	8	5	-	-	5.7	10	5.7	6.8
S-32071	7	13	9	5	11	4	-	-	7.7	10	7.7	8.8
S-32072	3	18	5	9	-	-	-	-	4.9	10	4.9	6.3
S-32074	6	5	8	3	10	3	-	-	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32075	10	10	12	6	14	5	-	-	10.0	10	10.0	10.3
S-32076	3	17	5	6	-	-	-	-	4.3	14	3.4	4.8
S-32078	4	24	6	13	-	-	-	-	n.a.[1]	14	5.8	7.0
S-32082	7	11	9	7	11	6	-	-	7.5	10	7.5	8.1
S-32083	7	19	9	10	11	6	-	-	9.0	10	9.0	9.5
S-32085	5	11	7	7	-	-	-	-	5.7	10	5.7	6.8
B (Second group)												
S-32572	7	44	9	9	-	-	-	-	8.9	14	8.6	10.4
S-32573	11	13	14	8	17	5	-	-	12.8	7	15.0	13.9
S-32575	13	22	15	13	17	4	-	-	15.6	10	15.6	15.0
S-32576	8	8	10	6	12	5	-	-	n.a.[1]	7	9.0	9.5
S-32577	6	10	8	8	10	5	-	-	6.0	10	6.0	6.8
S-32578	4	21	6	14	8	6	-	-	7.0	14	6.0	7.2
S-32579	8	12	10	8	-	-	-	-	9.0	10	9.0	10.1
S-32582	13	7	16	4	19	2	-	-	n.a.[1]	10	n.a.[1]	n.a.[1]
S-32583	11	12	14	4	17	2	-	-	11.7	10	11.7	11.3
S-32584	3	28	5	4	-	-	-	-	4.5	14	4.2	5.5

[1] n.a. — not available from data supplied.

[2] Obtained by extrapolation

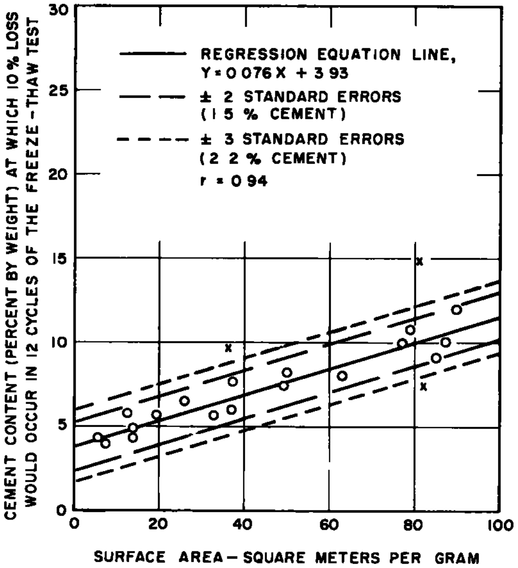

Figure 3. Cement content (percent by weight) at which 10 percent loss would occur in 12 cycles of the freeze-thaw test vs surface area of soils (first group.)

It was thought that expressing the cement requirement in a different way would obviate these difficulties. Strictly for purposes of correlation, in place of the recommended cement requirement, it is proposed to use for all soils that cement content (in percent by weight) at which 10 percent loss occurs in the freeze-thaw test. This places the cement content on a weight basis as is the surface area; it eliminates the bias due to different limits of allowable loss for different soil groups by placing all of the soils on a uniform basis of 10 percent loss; it eliminates personal judgment factors; and when the cement contents are expressed to the nearest 0.1 percent, it eliminates bias due to excessive rounding of the values.

This expression of cement requirement was obtained by the following procedure, as illustrated in Figure 2: For the several freeze-thaw specimens prepared from the same soil at various cement contents, a plot was made of the actual test loss vs cement content, and the points were connected by straight lines. The cement content at which 10 percent loss would occur was then read directly from this plot, by interpolation if necessary. The actual loss data and corresponding cement contents for all the soils are given in Table 2.

The relation between this new cement factor and surface area was investigated, with results as shown in Figure 3. The analysis showed a distinct and marked improvement in the degree of correlation existing, the correlation coefficient r being 0.94, which demonstrates both the validty of the original hypothesis that cement requirement and surface area are closely related, and also, that the disturbing factors discussed above have been largely avoided by this method of estimating the required cement content.

The regression equation appropriate for predicting the cement content by weight at which 10 percent loss occurs was calculated as:

$$Y = 0.076 \text{ (surface area)} + 3.93$$

The standard error of estimate from this equation is 0.74 percent cement. It should be noted that for several of the samples, the available freeze-thaw test data could not be used to obtain the test value of the cement content at which 10 percent loss would occur, except by questionable extrapolation; these samples have accordingly not been included in this correlation. In addition, three samples for which data were available (Nos. S-32056, S-32062, and S-32082) did not fit the correlation. Since these were more than three standard errors from the regression equation line, it is statistically valid to discard them from consideration on the grounds that they presumably do not belong to the same statistical population as the remainder of the samples. These three samples will be discussed later in the report.

A sufficiently close correlation has thus been established to permit prediction of the cement content at which 10 percent loss occurs from surface area measurements. It now remains to develop a procedure to convert the cement content so predicted for an individual soil, back to an estimated cement requirement based on the specific freeze-thaw loss allowable for soils of its class. For those soils where a maximum of 10 percent loss is allowed (A-2-6, A-2-7, A-4, and A-5 soils), no adjustment of the predicted value is of course necessary; for those (A-6, A-7) having a maximum allowable loss of 7 percent, the predicted cement content would be increased; similarly for soils (A-1, A-2-4, A-2-5) where loss up to 14 percent is allowed, the predicted cement content would be decreased. By examination of the loss data, the appropriate corrections were estimated to be +2.0 percent, and -0.7 percent, respectively. The corrected cement contents by weight can then be converted to a predicted cement requirement by volume through the use of the formula previously listed.

The above procedure was followed to obtain predicted cement requirements in percent by volume, which were then compared with the cement requirement (by volume) computed directly from the freeze-thaw test results. Agreement for Group A samples (Nos. S-32044 through S-32085) was only reasonably good. The coefficient of correlation between the predicted and the test cement requirements was r = 0.87, and the standard error of estimate was 1.3 percent cement.

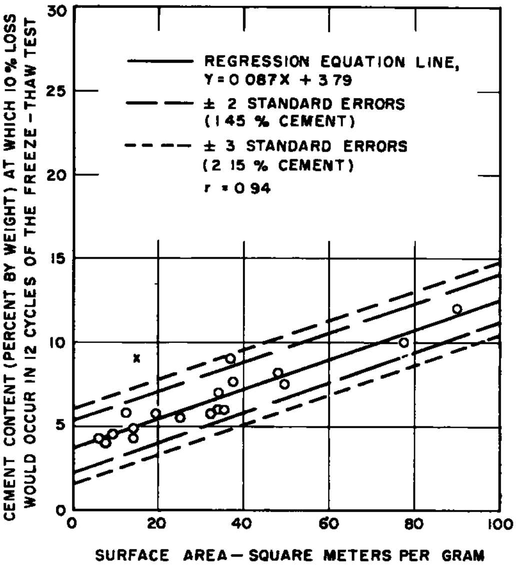

Figure 4A. Soils having less than 45 percent silt: cement content (by wt) at which 10 percent loss would occur in 12 cycles of the freeze-thaw test vs surface area.

Summarizing the results to this point, it has been shown that:

1. A definite though not very precise correlation exists between surface area and the cement requirement (percent cement by volume) actually recommended by the P.C.A. for this first group of plastic soils.

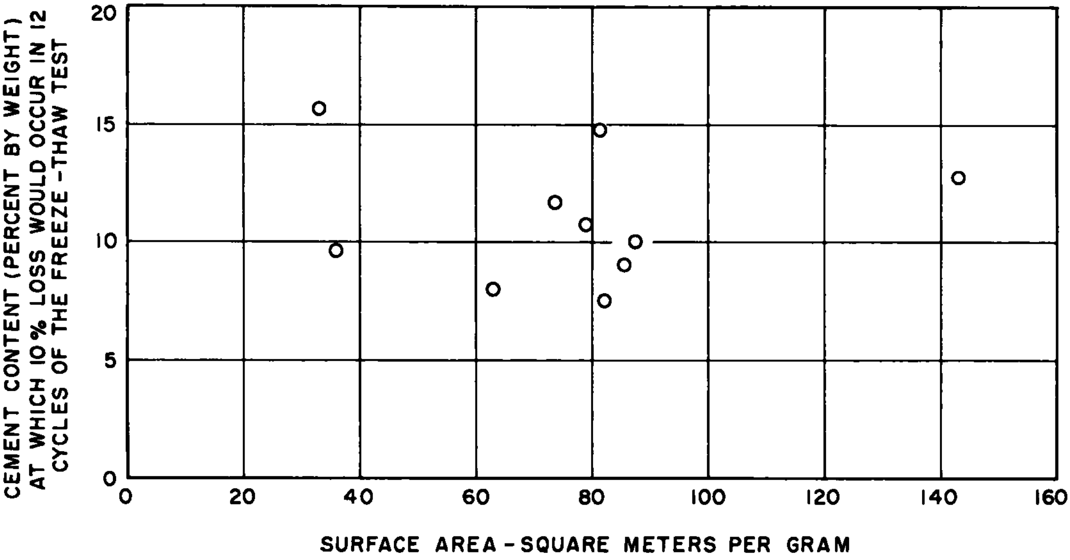

Figure 4B. Soils having 45 percent or more silt: cement content (by wt) at which 10 percent loss would occur in 12 cycles of the freeze-thaw test vs surface area.

TABLE 3

COMPARISON OF CEMENT REQUIREMENTS PREDICTED FROM SURFACE DATA WITH CEMENT REQUIREMENT FROM FREEZE-THAW TEST DATA

B. P. R. soil No.	AASHO classification	Surface area m²/g	Regression equation prediction of cement content at which 10 percent loss occurs % by wt	Corrections for soils where 7 percent or 14 percent loss is allowed % by wt	Corrected prediction of cement requirement % by wt	Corrected prediction of cement requirement % by vol	Cement requirement computed from freeze-thaw test data % by vol	Deviation of predicted from test cement requirement % by vol
Category 1 - Plastic soils, silt content less than 45%								
S-32044	A-2-4(0)	12.5	4.9	-0.7	4.2	4.9	5.7	-0.8
S-32045	A-6(7)	90	11.6	+2.0	13.6	13.1	13.2	-0.1
S-32046	A-6(2)	49.5	8.1	+2.0	10.1	10.6	10.4	+0.2
S-32053	A-2-6(1)	48	8.0	-	8.0	8.6	8.8	-0.2
S-32055	A-4(8)	37.5	7.1	-0.7	6.4	7.6	5.4	+2.2
S-32060	A-1-b(0)	14	5.0	-0.7	4.3	5.9	5.0	+0.9
S-32063	A-7-6(14)	77.5	10.5	+2.0	12.5	12.4	12.6	-0.2
S-32064	A-1-b(0)	75	4.4	-0.7	3.7	5.0	4.8	+0.2
S-32065	A-2-4(0)	25.5	6.0	-0.7	5.3	6.5	7.0	-0.5
S-32068	A-2-4(0)	28.5	6.3	-0.7	5.6	6.8	8.7	-1.9
S-32070	A-4(1)	19.5	5.5	-	5.5	6.6	6.8	-0.2
S-32071	A-4(4)	37.5	7.1	-	7.1	8.5	8.8	-0.3
S-32072	A-2-6(0)	14	5.0	-	5.0	6.4	6.3	+0.1
S-32976	A-2-4(0)	6	4.3	-0.7	3.6	4.8	4.8	0.0
S-32078	A-2-4(0)	20	5.5	-0.7	4.8	5.8	7.0	-1.2
S-32085	A-4(0)	32.5	6.6	-	6.6	7.7	6.8	+0.9
S-32576	A-6(6)	36	6.9	+2.0	8.9	9.5	9.5	0.0
S-32577	A-4(1)	34	6.8	-	6.8	7.7	6.8	+0.9
S-32578	A-2-4(0)	34	6.8	-0.7	6.1	7.3	7.2	+0.1
S-32579	A-4(0)	37	7.0	-	7.0	8.1	10.1	-2.0
S-32584	A-2-4(0)	9	4.6	-0.7	3.9	5.1	5.5	-0.4
S-32572[1]	A-2-4(0)	14.5	5.1	-0.7	4.4	5.6	10.4	-4.8[1]
Category 2 - Plastic soils, silt content 45% or higher								
S-32047	A-6(9)	63	9.3	+2.0	11.3	11.4	9.7	+1.7
S-32051	A-4(8)	79	10.7	-	10.7	10.8	10.9	-0.1
S-32056	A-4(8)	36	6.9	-	6.9	7.7	10.5	-2.8
S-32062	A-7-6(11)	81	10.8	+2.0	12.8	11.7	13.7	-2.0
S-32075	A-4(8)	87.5	11.4	-	11.4	11.6	10.3	+1.3
S-32082	A-4(8)	82	10.9	-	10.9	11.4	8.1	+3.3
S-32083	A-4(8)	85.5	11.2	-	11.2	11.6	9.5	+2.1
S-32573	A-7-6(15)	143	16.2	+2.0	18.2	16.2	13.9	+2.3
S-32575	A-4(8)	33	6.7	-	6.7	7.1	15.0	-7.9
S-32583	A-4(8)	73.5	10.2	-	10.2	9.8	11.3	-1.5

[1] Deviant sample, this was discarded in computing the regression equation.

2. The correlation was greatly improved by using instead of the recommended cement requirement by volume, the cement content by weight at which an arbitrary figure of 10 percent loss would occur in 12 cycles of the freeze-thaw test. The computed regression equation permitted satisfactory predictions of this value to be made from the surface area.

3. A procedure was developed for correcting this cement factor to predict the cement content (by weight) at which 7 percent or 12 percent loss would occur, and by the use of a given formula involving the density of the soil-cement product, this prediction could be converted to a volume basis.

4. Predictions thus made were in reasonable agreement with cement requirements derived directly from the freeze-thaw test data, the correlation between the two sets of values being 0.87.

Another group (sample Nos. S-32572 through S-32584) of soils received after work on the first group had been completed, provided an opportunity to check the validity of these results. These samples are described in Table 1 as Group B, and cement requirement test data for them are listed in Table 2. After the surface areas were determined for these soils, predictions of cement requirement (by volume) were computed, using the regression equation derived for the soils of Group A and the additional procedure outlined above. A comparison of these predicted values with cement requirements computed directly from the freeze-thaw test data indicated good agreement for some of the samples but considerable deviations for certain others. Upon examination of the engineering test data for Group B, it was noted that all but one of the deviant samples were very high in silt content, silt being taken as that portion passing the 200-mesh sieve and coarser than 0.005 mm. When the data for the samples of Group A were re-examined, it was found that here also high silt content was associated with relatively poor agreement between predicted and test results. In particular, it was noted that the three soils it was necessary to discard from the previous correlation were high in silt content. For such soils, the cement requirement is evidently governed by some property or properties other than surface area.

In order to verify this premise, the data of both groups of soils taken together were divided into two categories on the basis of silt content. Upon examination of the data, an appropriate dividing line appeared to be at a silt content of 45 percent. The correlation between surface area and the cement content (by weight) at which 10 percent

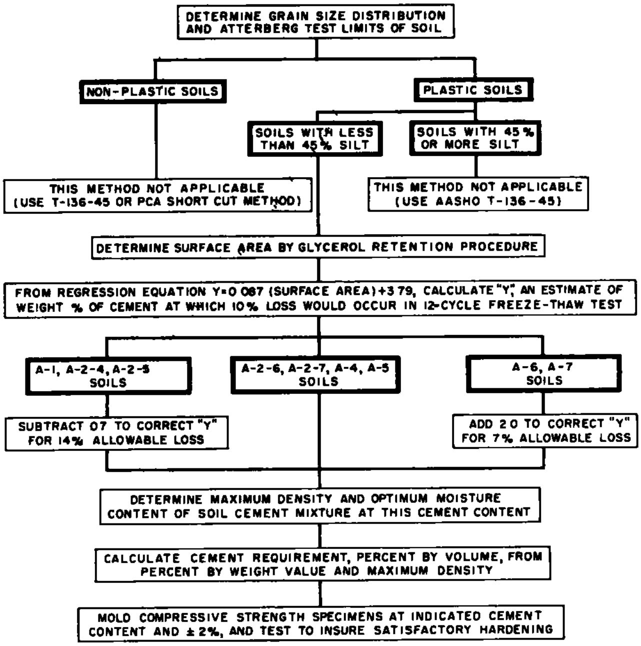

Figure 5. Flow sheet: short-cut method, using surface area to determine cement requirement of plastic soils containing less than 45 percent silt.

loss occurred in the freeze-thaw test was recomputed separately for each category. The category (22 samples) with silt content under 45 percent showed a high degree of correlation (correlation coefficient r = 0. 94) similar to that previously determined for the first batch alone. One deviant sample (No. 32572) did not fit the correlation, and since it was considerably more than three standard errors from the regression equation line, it was discarded. This sample is discussed later in the report. On the other hand, the category consisting of soils with silt contents of 45 percent or higher, showed essentially zero correlation (correlation coefficient r = -0. 03). The data of the two categories of samples are plotted separately in Figures 4A and 4B. The striking difference in correlation coefficients demonstrates clearly that the two sets of samples represent different populations; for the soils of lower silt content the cement requirement is essentially a function of surface area, as has previously been determined; but for soils of higher silt content there is little relation between the two.

The regression equation was then calculated for the soils of low silt content. It differs only slightly from the corresponding equation previously calculated for Group A samples alone. The new equation is: cement content (by weight) at which 10 percent loss occurs = 0. 087 (surface area) + 3. 79. The equation and the previously described procedure were then used to predict the values of cement requirement for these soils. These predictions are compared in the upper part of Table 3 with the cement requirements derived from the freeze-thaw test data. Agreement between the two sets of values is good (correlation coefficient r = 0. 92, standard error of estimate = 0. 9 percent cement), again with the exception of sample No. S-32572 previously noted as being a deviant sample. Neglecting this sample, the average deviation between predicted and test cement requirements was only 0. 6 percent cement. Among the remaining 21 soils, six were classified in the highly plastic AASHO groups A-2-6, A-2-7, A-6 and A-7. For all six, predictions were within 0. 2 percent cement of the test value.

The clay minerals present in the various samples were identified by x-ray diffraction techniques. The clay fractions were nearly all mixtures of two or more clay minerals, including among them montmorillonite, illite, kaolinite, chlorite, and vermiculite. Most contained at least a detectable amount of montmorillonite, and several were almost pure montmorillonite. None of the clays was principally illite or principally kaolinite, although several consisted largely of a chlorite-like clay mineral. From the evidence available, it seems that, in general, the cement reqirement is influenced by the surface area itself, without regard to the specific type of clay mineral from which the surface area is derived.

No definite explanation has been found for the behavior of sample No. S-32572 (point labeled x, Figure 4A), previously noted as strongly deviating from the regression equation line. The actual cement requirement is much higher than the predicted value. X-ray diffraction examination indicated that this soil consists primarily of finely divided calcium carbonate (caliche), much of it in the clay size-range. Further study is needed to determine the effect of this type of material on the cement requirement.

From the results obtained, a procedure for predicting cement requirements of plastic soils of less than 45 percent silt content was formulated. This is diagramed in Figure 5. For soils to which it can be applied, the procedure eliminates the time-consuming freeze-thaw tests. It would still be necessary, however, to prepare small specimens at and near the predicted cement requirement, and test them for compressive strength (or by other suitable means) to insure that adequate hardening was actually taking place. Furthermore, the predicted cement content should be modified by appropriate engineering judgment, to compensate for factors such as difficulty of adequate mixing in the field, and possible local variations in the soil materials.

It is recognized that these findings are based on only a limited number of samples, and are not necessarily applicable to all soils. For example, other workers have reported that certain types of soil organic matter strongly influence the cement requirement; although a number of the soils in this study were moderately high in organic matter, there was no evidence of appreciable effects on cement requirement. With a larger group of samples, soils containing such deleterious organic matter might have been encountered.

CONCLUSIONS

The following conclusions are drawn from this investigation:

1. For soils of measurable plasticity, a definite correlation exists between surface area, as measured by the glycerol retention procedure, and cement requirement (percent by volume) calculated from loss data of the freeze-thaw durability test and amended by other engineering considerations. However, this correlation is not sufficiently close to permit adequate predictions to be made directyly from the surface area values. The weakness of this correlation is considered to be due to such factors as:

a. The surface area is expressed on a weight basis in contrast to the volume basis of the cement requirement.

b. The different standards of allowable losses in the freeze-thaw test, 7, 10 or 14 percent, depending on the AASHO classification of the sample.

c. The use of a certain amount of engineering judgment in deriving practical recommendations for field use from the freeze-thaw data.

d. The rounding of the cement requirement to the nearest half-percent.

e. The presence in the group of samples studied of several soils high in silt content. Such soils have been shown to have cement requirements which do not correlate with surface area.

2. A very strong correlation is obtained when soils of higher than 45 percent silt are excluded from consideration, and the cement factor in the correlation is taken as the actual cement content by weight at which a 10 percent loss occurs in the freeze-thaw test, no allowance being made for AASHO class differences. The regression equation $y = 0.087$ (surface area) $+ 3.79$ can be employed to derive accurate predic-

tions of this cement factor from measurements of surface area by the glycerol retention method.

3. A suitable prediction of the conventional cement requirement, in percent by volume, can then be made by the following procedure:

   a. Modifying if necessary the cement factor predicted by the regression equatiin be adding 2.0 percent cement to adjust to the basis of a 7 percent allowable loss in the freeze-thaw test, or by subtracting 0.7 percent to adjust to the basis of an allowable loss of 14 percent.

   b. Converting the modified value to a percent by volume basis using the density of the soil-cement mixture.

In a comparison of cement requirement values obtained by test with values obtained by the use of this procedure, the average deviation for the group of samples used in this study was 0.6 percent cement by volume, and considerably less than this for the more highly plastic soils of the group.

4. Admittedly, these results were obtained from a restricted number of samples and the procedure should be checked further with a wider and more representative selection of soils. Furthermore, use of the surface area determination to predict cement requirements should be accompanied by compressive strength or other tests on small specimens made at and near the predicted cement requirement.

Appendix

Statistical Terms

The statistical terms employed in this study are as follows:

Correlation coefficient (r): A term which indicates the degree of association or relation between the measured values of one property and the corresponding measured values of another property, for a specified group of samples. This term varies from 1.0, indicating that a perfect functional relation exists and that one property could be predicted with absolute accuracy from knowledge of the other, to zero, which indicates a complete lack of relation between the two properties. If the relation is direct, i.e. if one property increases with increase in the other, the correlation coefficient is positive; if the relation is inverse, i.e., one property decreases with increase in the other, the correlation coefficient is negative. Generally, a correlation coefficient above 0.9 is required for the correlation to be good enough to permit predictions of one value from the other with a reasonable degree of accuracy.

Statistical Significance Level of Correlation Coefficient

This is a measure of the probability that so large a correlation coefficient as has been computed from the data could arise by pure chance sampling from a population in which there is in fact no correlation. A 0.1 percent or even a 1 percent significance level indicates that a correlation almost certainly does exist.

Regression equation: If a linear correlation exists between two properties of a group of samples, and a plot is made of property "Y" vs property "X" for all samples of the group, an array of scattered points results. A straight line may be drawn through the scattered points in such a way that it best fits the data, using as the criterion of "best fit" that the sum of the squares of the deviations of all of the points from the line is at a minimum. The equation of this line is called the regression equation, and its use permits the best estimate of values of property "Y" to be made from measured values of property "X".

Standard error of estimate (Sy): This is a measurement of deviation or degree of scatter of the points around the regression equation line. It has the same dimensions as the dependent variable, Y, and it provides an estimate of the uncertainty of the prediction of Y from X by means of the regression equation. If the normal distribution of errors hold, 19 out of 20 samples should fall within two standard errors of the regression equation line, and 997 out 1,000 within three standard errors.

REFERENCES

1. "Standard Method of Freezing and Thawing Test of Compacted Soil-Cement Mixtures." AASHO Designation: T 136-45, Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part II, Methods of Sampling and Testing (1955).
2. Catton, M. D, "Research on the Physical Relations of Soil and Soil-Cement Mixtures." Proceedings, Highway Research Board, Vol. 20, pp. 821-855 (1940).
3. Dyal, R. S. and Hendricks, S. B., "Total Surface of Clays in Polar Liquids as a Characteristic Index." Soil Science, Vol. 69, pp. 421-432 (1950).
4. Kinter, E. B. and Diamond, S., "Gravimetric Determination of Monolayer Glycerol Complexes of Clay Minerals." Proceedings, Fifth National Clay Conference, Urbana, Ill., in press (1956).
5. Diamond, S. and Kinter, E. B., "Surface Areas of Clay Minerals as Derived from Measurements of Glycerol Retention." Proceedings, Fifth National Clay Conference, Urbana, Ill., in press (1956).

Effect of Sulfates on Cement-Stabilized Clay

P. T. SHERWOOD, Road Research Laboratory, Department of Scientific and Industrial Research, United Kingdom

This paper describes an investigation into the effect of sulphates on the strength of cement-stabilized clays. It is found that sulphates have a detrimental effect on clay-cement mixtures, similar to that on concrete, which, in extreme cases, results in the complete disruption of the material. Sulphate contents up to 7 percent (as SO_3) of the dry weight of the clay had little effect on the compressive strength of the clay-cement when cured at constant moisture content but when more than 1.0 percent (as SO_3) of either magnesium or calcium sulphate was present in the clay, clay-cement mixtures disintegrated on immersion in water. The effect was even more pronounced when sulphate-free clay-cement was immersed in aqueous sulphate solutions; as little as 0.2 percent of sulphate in the surrounding water reduced the strength by more than 50 percent.

It is concluded that damage to cement-stabilized clay would not be less severe than that which would be experienced by an average quality concrete in the same circumstances.

● IN GREAT BRITAIN granular and light clay soils have been successfully stabilized with portland cement for the construction of road bases but few heavy clays have been so treated because of difficulties in mixing. The development in recent years of improved mixing machinery suggests that this limitation may soon be overcome, in which case it will be necessary to know whether there are in clays any chemical constituents affecting the hardening of cement, such as there are in some granular soils.

By analogy with the behaviour of concrete it might be expected that sulphates, which are widely distributed in British clays, would also have an adverse effect on soil-cement and that these would present a problem in the cement-stabilization of heavy clays.

This paper describes work which was undertaken at the Road Research Laboratory in Great Britain to find the effect of sulphates on the compressive strengths of hardened clay-cement mixtures.

OCCURRENCE OF SULPHATES IN SOILS

Sulphates usually occur in British soils in the form of the sparingly soluble calcium sulphate (gypsum) although sodium and magnesium sulphates are also encountered in certain areas. Bessey and Lea (1) have reviewed the distribution of sulphates in Great Britain and have noted that they usually occur only in clay soils, since gravels and sands are leached free from soluble constituents by percolating waters. For this reason the concentrations of sulphates in the surface soil layers are often lower than those in the subsoil so that concrete placed near the surface on a well-drained soil is likely to be free from attack. They also point out that the sulphate contents of soils and ground-water are subject to wide seasonal fluctuations and vary considerably over quite small areas; because of this measurements of the total sulphate content are of only limited value. The distribution of sulphates in a British clay is illustrated in Figure 1, which shows the variation with depth of the sulphate and calcium contents of a profile of Oxford clay; the top 4 ft has been leached free from calcium sulphate, which has accumulated in the middle layer. Tests have shown that only the sulphate-free layer can be successfully stabilized with cement.

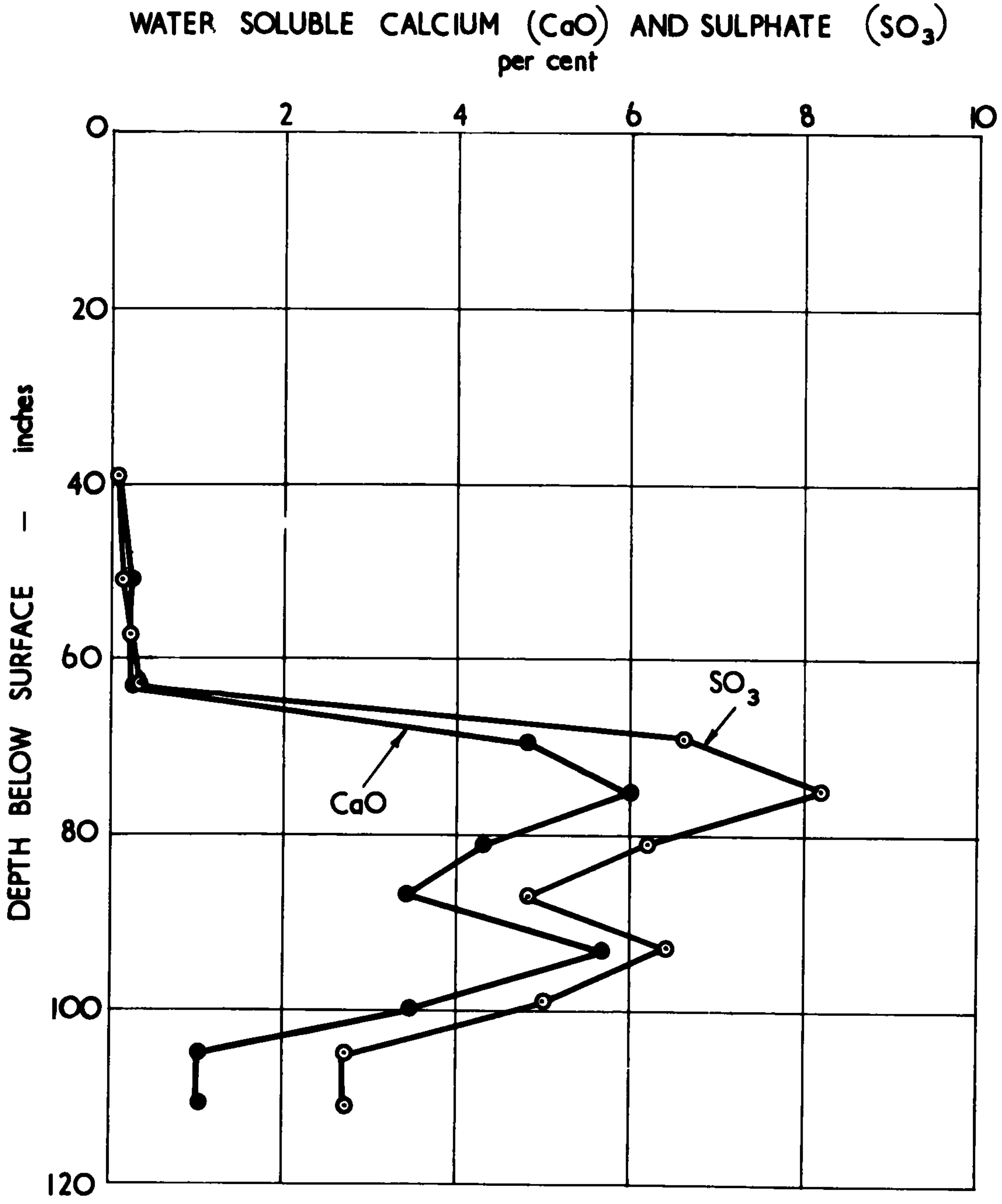

Figure 1. Variation with depth of sulphate and calcium contents for soil from an Oxford clay profile.

Under other climatic conditions the occurrence of sulphates may not be limited to clays and the lower soil strata. Moisture may percolate downwards during periods of heavy rainfall, but due to evaporation there will also be an appreciable upward movement of moisture in the dry season, with a resulting increase in the salt concentration in the surface strata. These conditions characterize the areas in which the Solonetz and Solontchak alkaline soils occur (2); e.g. Central Europe, the Southern United States of America and the Middle East.

POSSIBLE EFFECTS OF SULPHATES ON SOIL-CEMENT

There are three ways in which sulphates might be expected to affect the strength of soil-cement:

1. If sulphate-bearing ground-water flows through a soil-cement mixture, calcium sulphate can combine with the tri-calcium aluminate in the hydrated cement, producing calcium sulpho-aluminate according to the equation:

$$3\,CaO \cdot Al_2O_3 \cdot 12 \cdot H_2O + 3\,(CaSO_4 \cdot 2H_2O) + 13H_2O \longrightarrow 3\,CaO \cdot Al_2O_3 \cdot 3\,CaSO_4 \cdot 31H_2O \qquad (1)$$

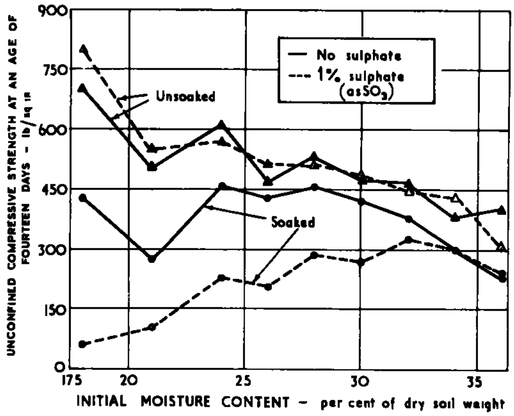

Figure 2. Effect of moisture content, sulphate content and immersion in water on the strength of a clay stabilized with ordinary portland cement.

This reaction is accompanied by a considerable increase in the volume of the solid material, and in concrete and mortar this has been shown to lead to the destruction of the hydrated cement matrix, and to loss of strength. The action of magnesium sulphate is even more marked in that it can also attack the calcium silicates in the hydrated cement, according to the equation:

$$3\,CaO \cdot SiO_2 \cdot n \cdot H_2O + Mg\,SO_4 \longrightarrow CaSO_4 \cdot 2H_2O + Mg\,(OH)_2 + SiO_2 \cdot n \cdot H_2O \qquad (2)$$

This reaction is also accompanied by a large increase in volume of the solid constituents, and the calcium sulphate formed is free to attack the tricalcium silicate according to equation 1. Since the reactants in equations 1 and 2, with the exception of magnesium sulphate, are only sparingly soluble in water, it would be expected that the action of sulphates would be more apparent when excess water can have access to the system and that the attack would result in a considerable expansion of the soil-cement. The effect of calcium (but not magnesium) sulphate can be partly overcome by increasing the ferric oxide content at the expense of the aluminium oxide content during the manufacture of cement, the product being known as "sulphate-resistant cement."

2. If a high proportion of water-soluble salts is present in a hardened soil-cement mixture two possibilities arise, both of which could lead to a reduction in strength when the soil-cement is immersed in water. In the case of a sparingly soluble salt the solid material will gradually dissolve leaving water-filled voids in the structure resulting in a reduction in the strength of the soil-cement. If the salt present is already in solution an osmotic pressure could be set up between the stronger salt solution in the soil-cement and the surrounding water; this pressure might be sufficient to cause some disruption of the soil-cement. The possibility of these circumstances occurring in practice is debatable, but cannot be completely ignored.

3. In arid areas it is known that a considerable upward movement of moisture can occur in soils. Soluble salts may be transported with this moisture and concentrated in the surface where the salts crystallize and disrupt the soil structure, creating what are known as "salt boils."

EXPERIMENTAL WORK

The experiments were devised to determine the effects of calcium and magnesium sulphate on cement-stabilized clay and to find in what manner and under what conditions these effects became apparent. This was done by comparing the strengths of the clay-cement when subjected to different forms of attack by the sulphates with the strengths of the clay-cement when unaffected by such attack. The strengths of the clay-cement mixtures were measured by the British Standard procedure (3) for determining the effect of immersion in water on the unconfined compressive strength of cylindrical specimens, 4-in. long and 2-in. diameter, of cement-stabilized soil. The British Standard test is a modification of the unconfined compressive strength test in which two identical specimens are prepared and cured at constant moisture content and temperature by coating them with paraffin wax and storing them at a temperature of 25 C. One specimen is left for 14 days; the other is left for 7 days after which time its wax coat is removed and the specimen is completely immersed in water for 7 days. The strengths of both specimens are determined at the end of the 14-day period.

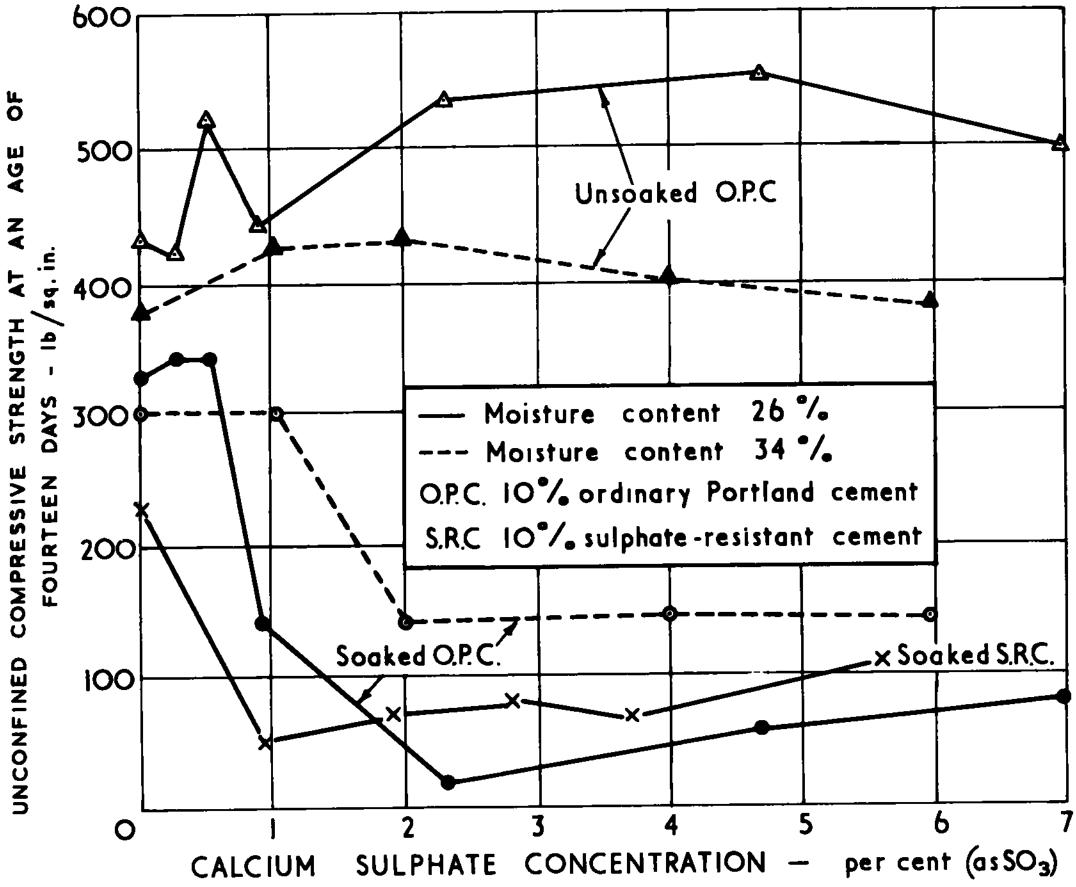

Figure 3. Effect of moisture content and calcium sulphate concentration on the strength of a clay stabilized with ordinary portland cement and sulphate-resistant cement.

The clay used for the work was a sample of sulphate-free boulder clay with a clay content of 30 percent (particles less than 0.002 mm in diameter), a liquid limit of 45 percent and a plastic limit of 26 percent. Soil-cement specimens were prepared by mixing the clay with the cement for 5 minutes and then mixing for a further 10 minutes with sufficient water to bring the mixture to the appropriate moisture content. The material was then compacted to an air-voids content of 5 percent.

To study the effect of the presence of sulphate in soil-cement mixtures the following investigations were made.

Effect of Sulphate and Moisture Contents

A study was first made of the effect of the presence of calcium sulphate in clay on the strength of the cement-stabilized clay when cured both normally and after immersion in water. Specimens for the water-immersion test were prepared from samples of the boulder clay at moisture contents ranging from 18 to 36 percent; 10 percent of ordinary portland cement was added and the specimens were compacted to an air-voids content of 5 percent. A similar set of specimens containing, in addition, 1 percent of calcium sulphate (as SO_3) was also prepared. The strengths of the specimens were measured either after normal curing for 14 days or after normal curing for 7 days plus immersion in water for 7 days.

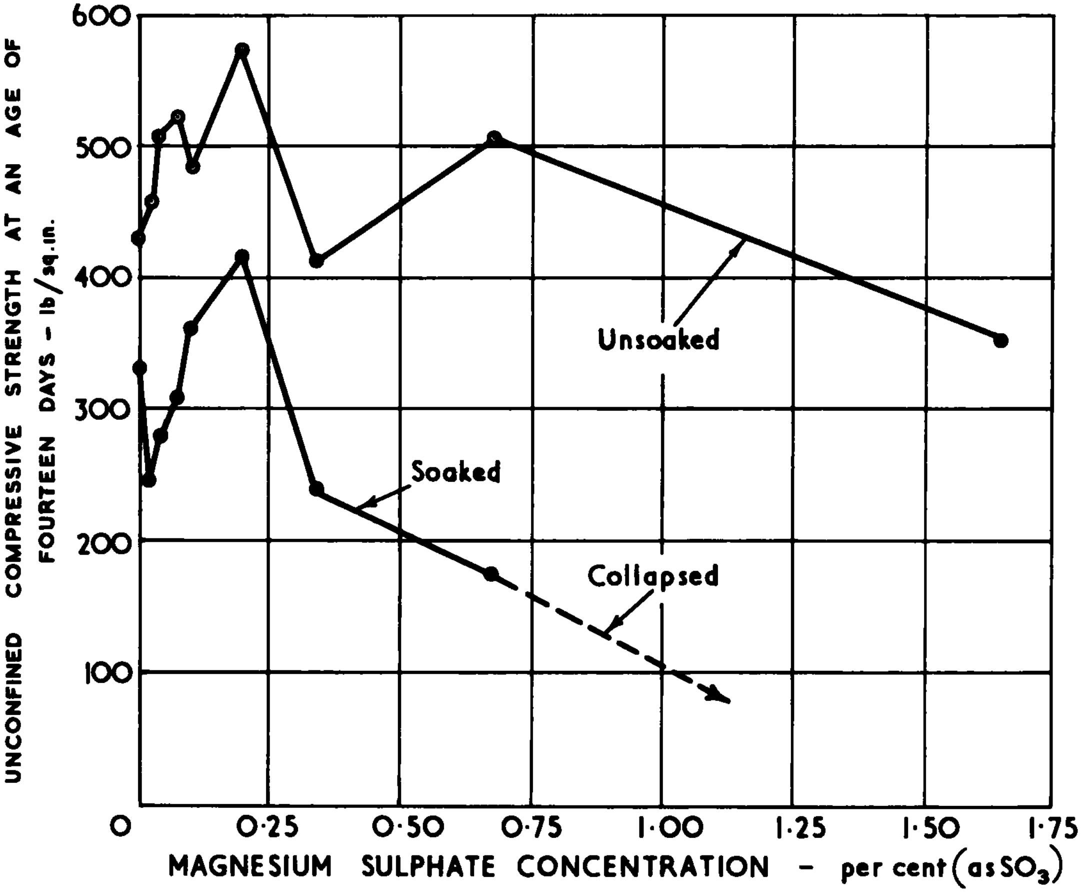

Figure 4. Effect of magnesium sulphate concentration on the strength of a clay stabilized with ordinary portland cement.

The results (Fig. 2) show that as the moisture content increased the strength of the normally cured soil-cement both with and without sulphate decreased and that the presence of the sulphate had no significant effect. Immersion in water, however, reduced the strengths at all moisture contents although to a smaller extent at the higher moisture contents. The presence of the sulphate resulted in significantly lower strengths than

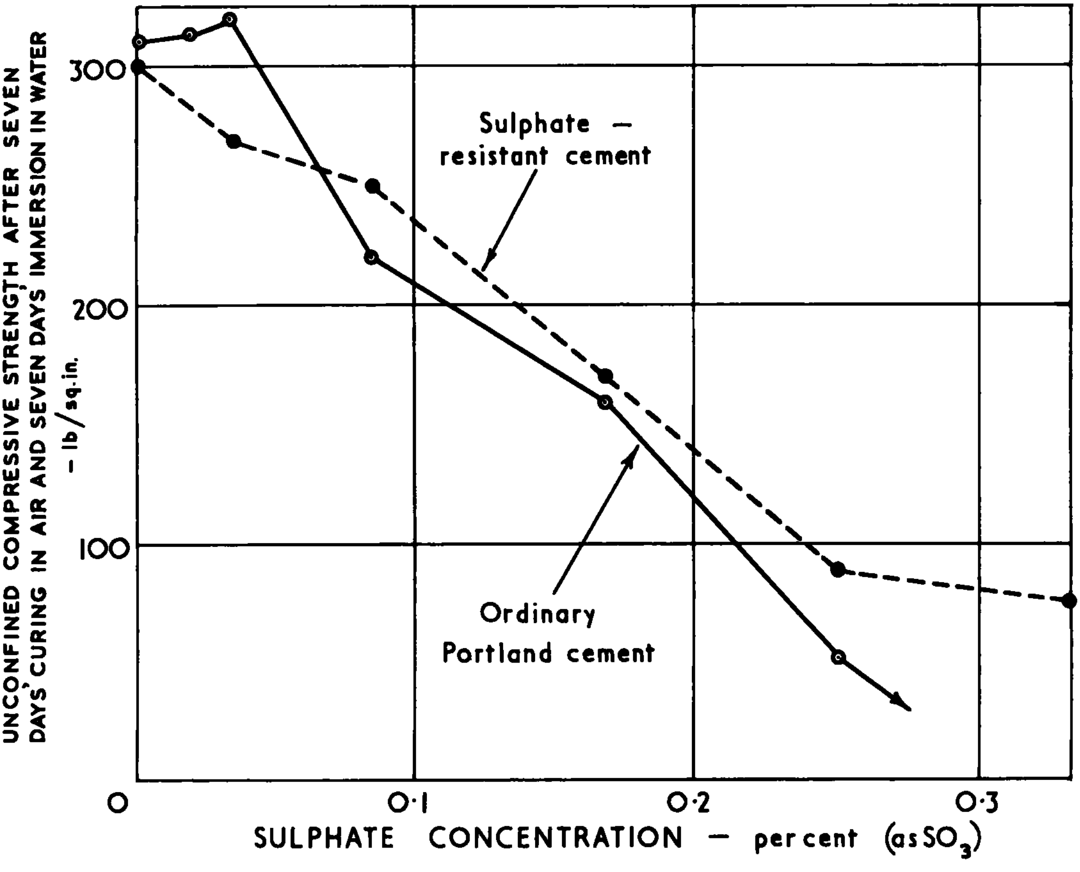

Figure 5. Effect of immersion in magnesium sulphate solution on the strength of boulder clay stabilized with ordinary portland cement and sulphate-resistant cement.

were obtained when it was absent, except when the moisture content of the clay was above 32 percent (i.e. some 6 percent above the plastic limit) (4).

Since an increase in the initial moisture content of the clay-cement appeared to have a beneficial effect, a series of tests was made in which two sets of specimens were prepared at moisture contents of 26 and 34 percent. The former is the moisture content at which the boulder clay would normally be stabilized (2 percent below the plastic limit of the clay-cement) while the latter is the highest moisture content at which it was found that specimens could be handled without deformation. Calcium sulphate in concentrations ranging from 0 to 7 percent (as SO_3) and 10 percent of ordinary portland cement were added.

The results (Fig. 3) confirm that at both moisture contents the presence of calcium sulphate in concentrations up to 7 percent (as SO_3) had little effect on the strengths of the normally cured specimens but after immersion the strengths of the specimens were greatly reduced at concentrations above 0.5 and 1.0 percent of SO_3 for initial moisture contents of 26 and 34 percent, respectively. Together with the previous results (Fig. 2) this suggests that as the sulphate content increases more moisture is required in the mixture to minimize the decrease in strength. However, the extent to which the moisture content of the mix can be increased is limited and, moreover, it seems likely that sulphate concentrations of the order of those measured in the profile would lead to loss in strength of the soil-cement mixture even if the moisture content of the mix was high.

In a further test to determine whether the effect of the calcium sulphate on the compressive strength of the clay-cement was reduced by the use of suphate-resistant cement, specimens were prepared which were identical with the foregoing except that sulphate-resistant cement replaced ordinary portland cement. The 14-day strength of this set of specimens was measured only after immersion. The strengths (Fig. 3) were not significantly higher than when ordinary portland cement was used. However, as sulphate-resistant cement has a lower content of tricalcium aluminate than ordinary portland cement and as this constituent is mainly responsible for early strength, higher strengths might possibly have been obtained if the specimens had been tested at a greater age.

A limited number of tests similar to those described above was carried out on specimens stabilized with 10 percent of cement but in which magnesium sulphate replaced calcium sulphate. The results (Fig. 4) show that the effect of the magnesium salt was more marked, in that the immersed specimens containing more than 0.7 percent of the salt (as SO_3) completely disintegrated.

Effect of Immersion of Soil-Cement in Magnesium Sulphate Solution

In practice it is possible that a stabilized sulphate-free clay might be constructed on a sulphate-bearing foundation. Experiments were therefore made to determine the destructive effect on a clay-cement mixture of contact with a sulphate-bearing solution.

Unconfined compressive strength specimens were prepared from samples of the boulder clay mixed with 10 percent of ordinary portland cement and 26 percent of water compacted to an air-voids content of 5 percent. After curing for 7 days at

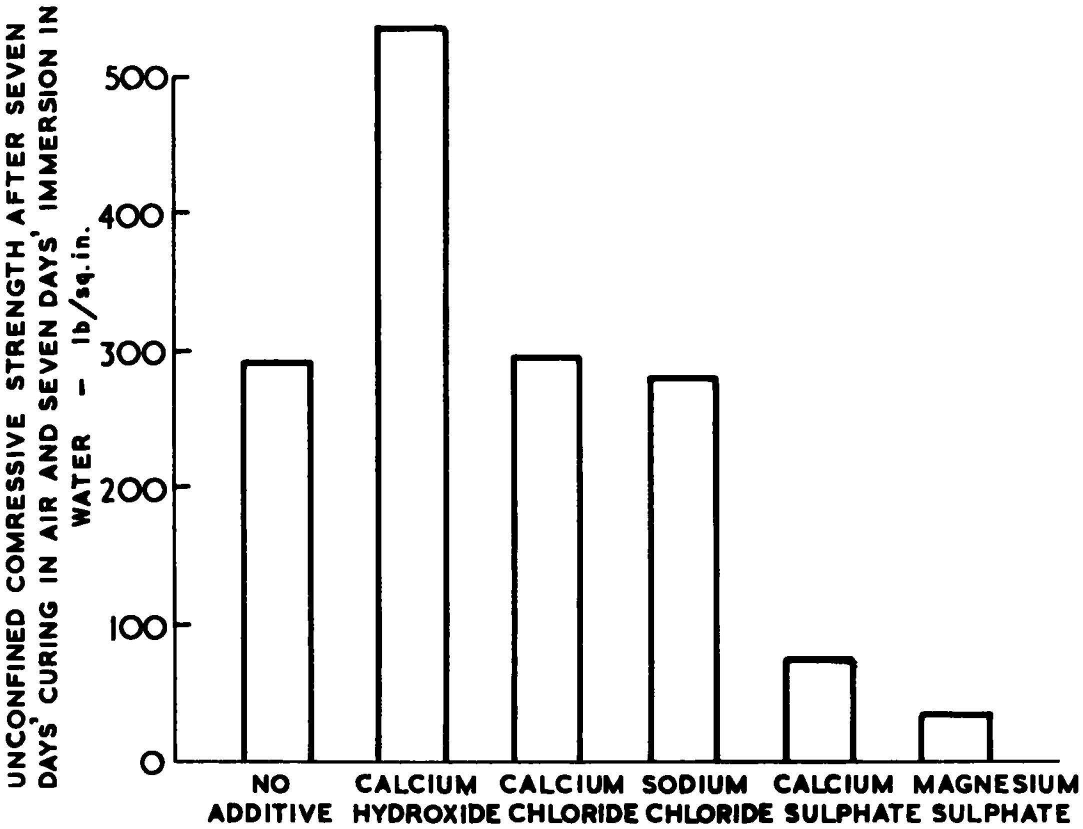

Figure 6. Effect of salts at equivalent concentrations on the strength of boulder clay stabilized with ordinary portland cement.

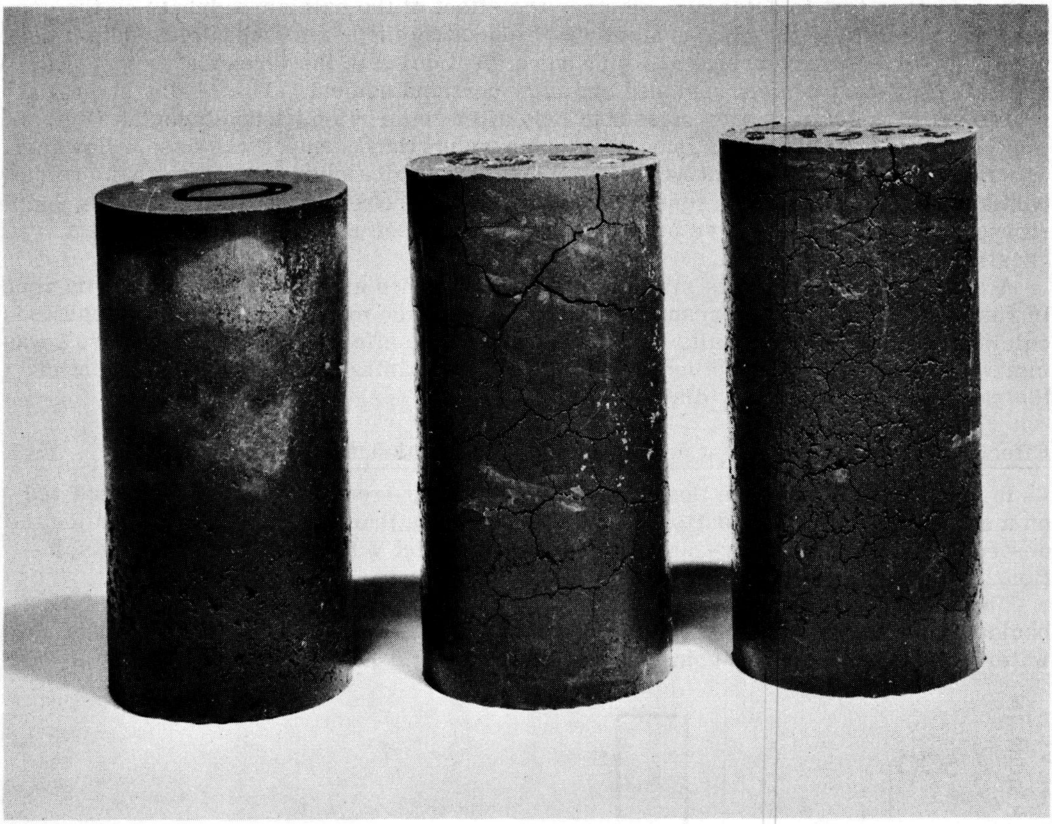

Figure 7. Effect of sulphates on soil-cement after immersion in water for seven days. (Left to right, control specimen, specimen + calcium sulphate, specimen + magnesium sulphate.

constant moisture content and temperature these specimens were immersed in solution of magnesium sulphate of different concentrations for a further 7 days, and their compressive strengths were than determined. A parallel experiment was made using sulphate-resistant cement. The results (Fig. 5) showed that when the concentration of sulphate in the water exceeded 0.05 percent of SO_3 significant reductions in strength were obtained, and when 0.20 percent was present the strength was reduced by 50 percent. In four out of the six specimens the sulphate-resistant cement gave significantly higher strengths after immersion in water than did ordinary portland cement.

Investigation of a Possible Osmotic Pressure Effect

Clay-cement specimens were prepared using boulder clay with 10 percent of ordinary portland cement and admixtures of calcium hydroxide, calcium chloride, calcium sulphate, magnesium sulphate and sodium chloride. The specimens were then subjected to the immersion test. The amount of salt added was sufficient to provide the equivalent weight in grams per litre of water in the soil-cement. With the calcium chloride, soldium chloride and magnesium sulphate all the salt present would be in solution and, as they were at equivalent concentrations and are all strong electrolytes, they would give rise to equal osmotic pressures. In the case of the sparingly soluble calcium hydroxide and calcium sulphate insufficient water was present for the salts to go into complete solution, and a large proportion of the salts would be present in the solid form; loss in strength would therefore be more likely to arise from replacement of the solid material by water than by osmotic pressure. The results show that only the specimens containing sulphates had lower strengths (Fig. 6) and exhibited signs of expansion and cracking (Fig. 7). It is concluded therefore that reduced strength is not due to internal stresses caused by osmotic pressures.

Effect of Partial Immersion in Water on Soil-Cement Containing Soluble Salts

To examine the effect on the strength of soil-cement of the crystallization of salts in the surface, caused by the capillary rise of moisture through a soil containing soluble salts, a soil-cement specimen was prepared from a soil from Abadan, Persia, by mixing with 10 percent of ordinary portland cement. This soil was chosen because it comes from an area where such crystallization effects are known to occur and also because it contained 1.2 percent of calcium sulphate (as SO_3) and 1.5 percent of sodium chloride (as Cl). The soil-cement specimen was allowed to cure at constant moisture content and temperature for 7 days and was then placed in water to a depth of a half-inch for several weeks during which time moisture was absorbed through the base of the cylinder and evaporated from the upper surfaces, causing a concentration of the soluble salts in the top of the specimens. Figure 8 shows the appearance of a specimen after two weeks; the upper surface had disrupted owing to crystal formation and had become so loose that the surface layers of the soil-cement could be easily scraped off with a penknife.

DISCUSSION OF THE EFFECT OF SULPHATES ON SOIL-CEMENT

The deleterious effect of sulphate-bearing ground-water on structures containing cement is well known, although cases where concrete road slabs are attacked are not of frequent occurrence in the United Kingdom. In the case of concrete building structures below ground level the problem has been dealt with in a publication from the Building Research Station (5), which classifies clay soils and ground-waters into the following three groups with respect to the risk of sulphate attack:

1. Sites with low sulphate content:

Ground-water	30 parts per 100,000 of SO_3
Clay	0.2 percent SO_3

2. Sites of moderate severity with regard to sulphate attack:

Ground-water	30 - 100 parts per 100,000 of SO_3
Clay	0.2 - 0.5 percent SO_3

3. Sites with high risk of sulphate attack:

Ground-water	100 parts per 100,000 of SO_3
Clay	0.5 percent SO_3

The results obtained in the investigations described in this paper indicate that

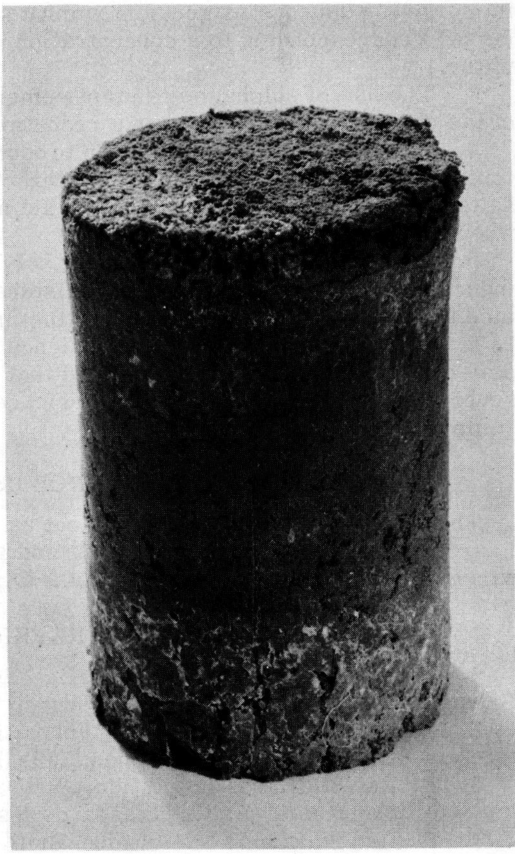

Figure 8. Specimen of soil-cement made from soil containing soluble salts, showing disruption of the top surface caused by crystal formation.

there is a serious risk of the deterioration of clay-cement mixtures in contact with contaminated clay or ground-water if sulphate is present in concentrations of the same order as those in groups 2 and 3 of the above classification. Sufficient site experience is not yet available, however, to indicate whether the numerical values quoted are directly applicable to sites where clay-cement road bases are to be used, and further research is required on this problem.

Damage arising from the crystallization of salts would be expected wherever salts (not necessarily sulphates) were present and the climatic conditions were such (i. e. long dry periods) as to cause the salts to move with the moisture into the surface and crystallize. The damage caused by this action can be minimized by sealing the surface so that excessive evaporation losses do not occur.

CONCLUSIONS

The following conclusions have been drawn from the work described:

1. In experiments in which calcium and magnesium sulphates were added to salt-free clay, the presence of magnesium sulphate in the clay in concentrations above 0.75 percent (as SO_3) by weight of the soil resulted in disintegration when the cement-stabilized clay was immersed in water. Calcium sulphate in concentrations greater than 1 percent (SO_3) considerably reduced the compressive strength, but the reduction was lessened by increasing the moisture content of the mix.
2. The strength of a clay-cement mixture was reduced by more than 50 percent when immersed in aqueous magnesium sulphate solutions of 0.20 percent concentration (as SO_3) corresponding to a concentration of 2,000 parts per million of SO_3 in ground-water.
3. The use of sulphate-resistant cement instead of portland cement as the stabilizer of the clay soil did not result in a reduction in the action of the sulphates.
4. The effect of the sulphates in producing a loss in strength of the clay-cement on immersion in water was due to the interaction of the sulphate and the cement, and salts which did not react with cement did not seriously affect the strength of soil-cement when it was totally immersed in water.
5. In the case of a clay containing soluble salts a loss in strength was obtained by inducing an upward movement of moisture so that the salts were deposited in the surface where they caused disruption of the material.
6. There is little practical experience of the attack by solutions of sulphates on soil-cement, but it would be expected that the damage to soil-cement would be not less severe than that which would be experienced by an average quality of concrete in the same circumstances.

ACKNOWLEDGMENTS

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REFERENCES

1. Bessey, G. E., and Lea, F. M., "The Distribution of Sulphates in Clay Soils and Ground-Water." Proc. Inst. Civ. Engrs, 2 (1), 159-81 (1953).
2. Jenny, H., "Factors of Soil Formation: A System of Quantitative Pedology, McGraw-Hill, New York and London (1941).
3. British Standards Institution, British Standard 1924, 1957. "Methods of Test for Stabilized Soils." British Standards Institution, London (1957).
4. British Standards Institution, British Standard 1377, 1948, "Methods of Test for Soil Classification and Compaction." British Standards Institution, London (1948).
5. Building Research Station, "Concrete in Sulphate Bearing Clays and Ground-Waters." Building Research Station, Digest No. 31, H.M. Stationery Office, London (1951).

Cementation of Soil Minerals with Portland Cement orAlkalis

R. L. HANDY, Engineering Experiment Station, Iowa State College, Ames

Although cementation of mineral surfaces has obvious importance in coarse-graded mixtures such as concrete, surface reactions become even more important in finer-grained mixtures such as soil-cement because of the far greater surface areas involved.

Unexpected long-term hardening of certain soil-cement mixtures has led to a hypothesis of hardening involving not only changes in the hydrating cement gel, but also changes within the surface layer of the mineral grains. The hypothesis follows Weyl's proposals of polarization or readjustment of ions near the surface of a solid to partly compensate for the unbalance in forces occurring at that surface. According to theory, such polarization "screening" should reduce the potential of a surface for chemisorption, or chemical bonding to other ions.

Once a mineral comes in close contact with an inorganic cementitious gel, bonds developing between the mineral surface and the gel should tend to reduce the unbalance in forces at the mineral surface. Because the cause of initial polarization is less, the polarization may reasonably be expected to become less. Reduced polarization then allows improved chemical bonding at the surface of the mineral, which in turn allows a further reduction in polarization screening. Thus a bond should show a gradual increase in strength over a period of time.

The data show a logarithmic relationship between compressive strength of soil-cement and curing time. Knowledge of the relative polarizability of various minerals may correlate with the rate of strength gain.

● **CEMENTATION CAN BE IMAGINED** as a combination of mechanical bonding of the cement to rough mineral surfaces, plus chemical bonds developing between the cement and the mineral surfaces. The latter process becomes more important as materials become finer grained; surfaces tend to be smoother, and more surface area is available. Portland cement stabilization of fine-grained soils thus emphasizes the chemical aspect, although answers obtained here would be expected to apply to a lesser extent in the cementation of coarse materials.

The present paper offers a hypothesis whose only pretense is perhaps to be reasonable in context, albeit rather evasive in proof.

BONDING IN MINERALS

Minerals are crystalline; that is, they have an orderly internal arrangement of atoms. The bonding between atoms is primarily ionic in character, though covalence, or electron sharing, does exist. Purely ionically bonded atoms retain their respective electrons and are attracted electrostatically because of the net plus and minus charges. These are not true bonds in the strict sense of the word, they are forces. Adjacent oppositely charged ions may be said to "screen" each other, since they act to balance out each other's forces.

Evidence shows that these two types of bonds (ionic and covalent) are transitional, and both can exist in any one linkage. Pauling (12) working with halides, developed

TABLE 1

ELECTRONEGATIVITIES OF ELEMENTS AFTER PAULING (12)

Element	Electronegativity
H	2.1
C	2.5
O	3.5
Na	0.9
Mg	1.2
Al	1.5
Si	1.8
P	2.1
S	2.5
Cl	3.0
K	0.8
Ca	1.0
Fe	1.7

expressions relating the percentage of ionic character in a bond to the "electronegativity" of elements making up that bond. Electronegativity is the ability of atoms in a molecule to attract electrons. Electronegativities (Table 1) show a close relation to position of the element in the Periodic Table. A curve relating these to percent ionic character is presented in Figure 1. The greater the difference in electronegativity between two elements, the greater the percentage of ionic character existing in their bond.

A measure of ionic vs covalent bonding, as well as actual positions of atoms, is obtained by plotting electron densities from various crystal planes by techniques involving three-dimensional Fourier analysis of X-ray diffraction intensities. In an ionic crystal such as sodium chloride the electron density falls to zero between adjacent positive and negative ions. In a covalent crystal such as diamond the electron density between adjacent atoms does not fall so low (about 1.7 per cubic Angstrom in the case of diamond). In intermediate crystals the minimum electron density is intermediate (5).

For the most part the percentages of ionic character in various bonds occurring in minerals have not been measured, and one must relay on the relationship of Pauling (Fig. 1) to obtain average values. Calculated values important in minerals are as follows.[1]

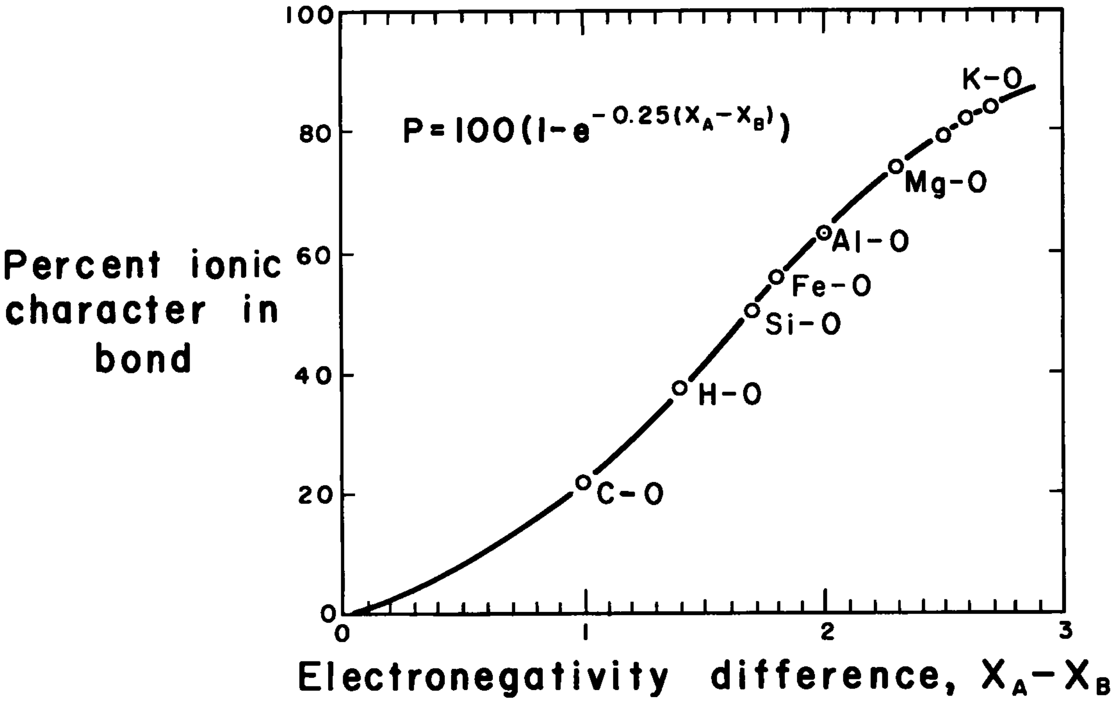

Figure 1. Percent ionic character in bonds. Calculated after Pauling (1).

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[1] $P = 1 - e^{-\frac{1}{4}(X_A - X_B)^2}$ 100. A modified treatment by Hannoy and Smyth, described in Coulson (2), gives $P = 16(X_A - X_B) + 3.5(X_A - X_B)^2$. Neither relationship is perfect because of difficulties in evaluating effects of resonance.

Of these only H-O and C-O are predominantly covalent in character. These occur as distinctive units in minerals, H-O as OH or in H_2O; C-O as CO_3 groups in the carbonate minerals. The Si-O bond is borderline.

So far the picture is relatively simple, and calculation of ionic character is uncomplicated if one anion and one cation combine to make up a molecule. But minerals often contain several kinds of cations, for example orthoclase feldspar, $KAlSi_3O_8$, and anions are usually shared by two or more cations. Gruner (6) in his application of Pauling's theories to mineral stability, simplified the problem by calculating the average electronegativity of all cations and adding a correction "bridging factor" for sharing of the O ions. Such a simplification is practical because in most minerals all cations are bonded to oxygen.

TABLE 2

Bond	Percent ionic character
K - O	84
Na - O	82
Ca - O	79
Mg - O	73
Al - O	63
Fe - O	56
Si - O	51
H - O	39
C - O	22

In discussing bond type, Pauling refers to "polarization," or electron shell deformation effects. Covalent bonds occur only when polarizability of the participant atoms allows the electron swarms to be distorted.

POLARIZATION AND SCREENING OF MINERAL SURFACES

Mineral grain interiors are essentially ions captured in a three-dimensional network of electrical forces. At a surface the continuity is interrupted and force patterns are disrupted. Surface tension is one result; the partial reorientation causes attractive forces to be diverted into the plane of the surface.

In a solid, several mechanisms may operate to reduce the energy in a surface. One of these, sorption of ions and molecules, is the goal of cementation. A second mechanism suggested by Weyl (16) is polarization of the surface ions, or distortion of their electron swarms. This is illustrated in Figure 2(b). Unbalanced forces are represented by arrows, and polarization tends to reduce the unbalance. Polarization is believed to be more pronounced in anions than in cations because of the larger diameter of electron swarms about anions. Thus polarization tends to create a negatively charged surface on a crystal. The extent to which polarization will take place varies depending on the nature of the ions and coordination of the bond. Large ions polarize more easily; small ions not at all.

A third process suggested by Weyl for reducing surface energy is distortion of the surface layer, the larger ions becoming slightly displaced and tending to "screen" the smaller cations. As seen in Figure 2(c), this would result in the formation of an electrical double layer in the surface of a mineral, the outer layer being negative. The difference from polarization is mainly one of degree, in that distortion in the surface layer also disturbs the force pattern of ions deeper in the crystal, and distortion screening effects probably extend a finite depth before dying out. The distortion mechanism may be analogous to the formation of an "amorphous" Beilby layer in metals.

A fourth mechanism important in stabilization of colloids is the formation of an electrical double layer by predominant adsorption of anions to screen the exposed cations. The process is here considered as part of chemisorption.

Comparison of Mechanisms

Polarization of surface ions. Of the above surface phenomena, polarization would be expected to take place on the crystal surfaces containing large ions having readily polarizable electron swarms. Polarization involving only the surface ions probably takes place rather quickly. Screening should be most effective when the number of cations can then be screened more easily. Polarization would presumably be decreased by covalent bonding tending to tighten the electron swarms around the affected ions.

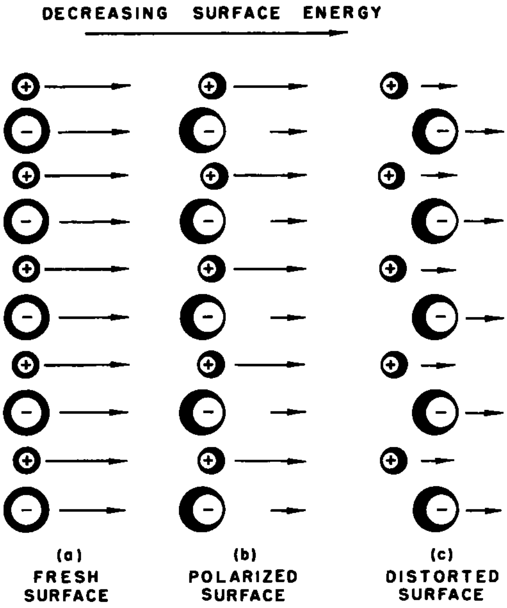


Figure 2. Polarization (b) and distortion screening (c) in a 100 NaCl surface. After Weyl (4).

Analysis of these factors suggests that silicates in general are only faintly polarizable in their surface layers.

Structural screening. The fact that surface ions are limited in their adjustment by the ions underneath does not mean that screening cannot occur; it means that ions underneath must move also. Eventually a surface zone of distortion may develop, each successive layer of ions being less out of place than the next one above. The sum of these distortions may add up to effective screening. X-ray diffraction of fine quartz powders reveals a strongly distorted zone extending 0.3 to 0.5 micron in from the surface. That these adjustments take time is evidenced by the fact that freshly ground quartz is chemically very active; it can lower the pH of water to below 5, presumably by adsorption of OH ions to cover unscreened Si^{+4} sites, and it can cause silicosis in man. The affinity of fresh quartz surfaces for anions is further indicated by presence of ozone in the grinding mill; O_2 covalent bonds are broken to supply oxygen ions and give a byproduct of $O + O_2 = O_3$ (16). Quartz

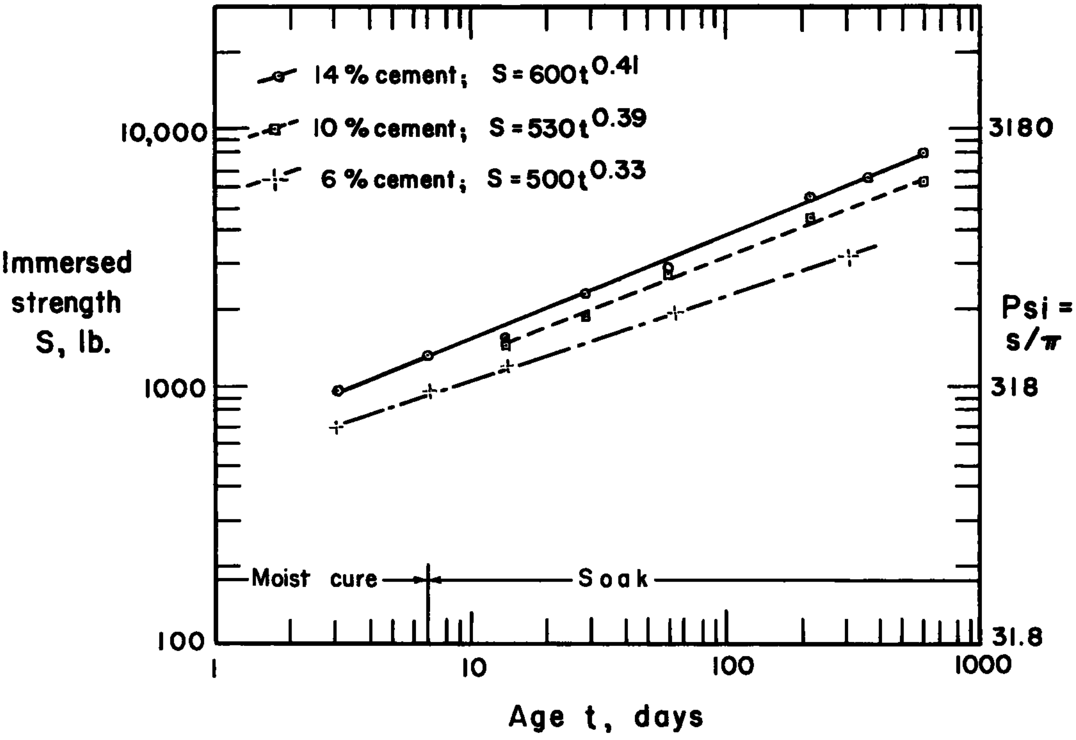


Figure 3. Relation between time and strength of 2-in. by 2-in. diameter soil-cement cylinders with Iowa loess sample 20-2. Samples were continuously soaked after initial 7 days moist curing.

which has aged a while is apparently less reactive and probably less dangerous.

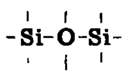
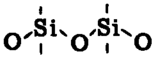
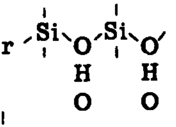
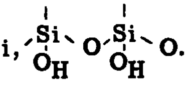
For reasons discussed later, quartz probably has an unusually high amount of surface zone distortion, and it is interesting to note the relation between quartz weathering and particle size. Quartz with its screened surface is ordinarily rather resistant to weathering, more so than the feldspars, yet in the clay size range the relative positions of quartz and feldspar are reversed (9). Probably in fine particles the distortion effects fading in from one side of a crystal are met by similar effects trying to fade in from the other. The result is less effective screening and a surface more susceptible to attack.

Chemisorption. Adsorption of ions takes place to satisfy bonding energies unshielded by either polarization of surface ions or the deeper structural screening. Even mild chemisorption reduces the surface energy. For example, a polar liquid such as water will temporarily satisfy some of the surface forces and cause a reorientation, reducing surface tension and decreasing the scratch hardness of many silicate minerals. Thus quartz grinds more easily when wet with water, and glass scribes more easily if the cutter is wet with water. The adsorption of OH ions, by reducing surface tension of the quartz, also contributes to solubility. Quartz and silica glass are much more soluble in alkaline than in neutral or acid conditions.

ALKALI REACTIONS

Alkali reactions, now well documented in concrete (13), proceed most rapidly with amorphous or glassy materials such as opal, opaline chert, and volcanic ash. The reactions are not generally recognized as proceeding with quartz. This serves to emphasize an important point—a structurally-screened surface zone is not randomly constituted, as is a glass. The random bonding in a glass contributes to "weak places" which make the surface highly susceptible to attack. Screening distortions, on the other hand, increase the resistance to attack. Screening would also be expected to occur in a glass, reducing the suitability for reaction. But because of the disorganized internal structure, screening may not shield cations symetrically, as in crystals.

CEMENTATION

Portland cement is an alkaline substance. Na_2O and K_2O are often present in minor amounts, the amount being limited to reduce alkali reactions with bad aggregates. $Ca(OH)_2$ is released on hydration. Hydroxl ions are thus available for immediate bonding to quartz. Polarization of the O ion may weaken the O-H bond and result in the loss of a hydrogen, but this is not known. Devore (4), in his application of Weyl's theories, considers OH ions to exchange for O ions in the surface of quartz, the displaced O ions being then bonded to the H. OH ions in this position are presumably more polarizable. The picture is then  $-\mathrm{Si}-\mathrm{O}-\mathrm{Si}-$ distorted to  $\mathrm{O}\diagup\mathrm{Si}\diagdown\mathrm{O}\diagup\mathrm{Si}\diagdown\mathrm{O}$ as oxygens move out. Adsorption to complete the screening is either  $\diagup\mathrm{Si}\diagdown\mathrm{O}\diagup\mathrm{Si}\diagdown\mathrm{O}\diagup$ (Devore) or a direct adsorption of OH onto partially covered Si,  $\diagup\mathrm{Si}\diagdown\mathrm{O}\diagup\mathrm{Si}\diagdown\mathrm{O}$.

Devore develops his theory of hydration to show how a quartz surface might assume the configuration of a feldspar or mica, thereby increasing the probability of intergrowth with those minerals.

In his study of bonding with cement pastes, Munger (10, 11) suggests that cement gel develops spontaneously on a mineral surface, bonding to the exposed oxygens, growing by polymerization of the SiO_4 groups, and incorporating free calcium ions into the linked SiO_4 superstructure. Bonding thus depends on the number of surface oxygens either in the ionic or covalent state. Munger correlates early bond strength

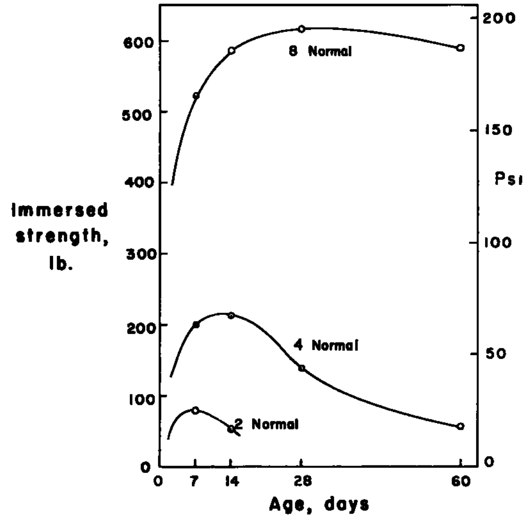


Figure 4. Strength of 2-in. by 2-in. diameter loess specimens compacted to standard Proctor and with various normality NaOH solutions substituted for added water (15 percent). Continuous moist cure, with 24 hour immersion prior to testing.

to the effective number of oxygen ions, and later bond strength to the degree of covalent bonding of oxygens. He believes covalent linkages strengthen as a result of the continuing polymerization.

Munger's theory would at first suggest that structural displacement screening would improve early cementation by increasing the exposed area of oxygen ions. On the other hand an unscreened or partially screened surface will immediately become coated by oxygens or OH ions from solution. Neither of these factors enter into Munger's calculation except indirectly, since both screening and effective ion calculations depend on kind and number of internal cations.

ADJUSTMENTS FOLLOWING CEMENTATION

The strength of soil-cement and concrete continues to gain through years, although data on mortars show that cement hydration has essentially stopped after about 24 months (2). Figure 3 shows the relationship between strength of a portland cement-stabilized silt and time, with strengths reaching almost 3,000 psi (80). Furthermore the increase in slope (the exponent on t) between 6 and 10 percent cement is significant, whereas the change from 10 to 14 percent cement is rather small. This suggests that above a certain minimum content of cement the hardening is not dependent on changes in the gel but reflects increased bonding to the mineral crystals. If the gel alone were affected there should be a closer relationship between slope and cement content. Also, the failures appear to involve breaks from the grain surfaces, not breaks through the grains, as sometimes occurs in coarse aggregates in concrete.

It would appear that as cementation proceeds, screening by polarization and distortion may change. Development of new bonds on an ionic surface should logically cause an immediate reorganization of the force pattern, the amount of the reorganization depending on the strength and number of cementation bonds. The net effect should be to reduce the surface disruption that caused screening, and to cause structural adjustments which would increase cementation by providing a more satisfactory atomic surface pattern. Should a previous nonstoichiometric surface adsorption be involved, diffusion of cations through the cement gel could make up the valence deficiency. The increased bond strength might then cause a further change in screening, which would give a further boost to bond strength. Thus an interdependence may exist whose effect is to put a time lag in the strengthening of cementation bonds.

The time necessary for such hypothetical adjustments to take place is unknown. If the polarization effects were only "skin-deep" as in Figure 2b, adjustments should be almost immediate, since they involve only the surface electron swarms. On the other hand if screening involves distortions extending tens or hundreds of ions into a crystal, with distortion of each successive layer depending on unbalanced forces set up by distortions in the last, time may become important.

The time required for adjustments to improve cementation should be longer than the time for structural screening of a fresh surface. Original screening was caused by a sharp discontinuity at the surface, whereas the opposing forces of cementation are at first comparatively weak. Their weak force pattern must be mirrored by an adjust-

TABLE 3

TENSILE STRENGTHS OF MORTAR BRIQUETS IN PSI[1]

Aggregate	Formula	52-hour moist cure		24-hour moist cure, 24-hour autoclave 150 C, broken at 52 hr		52-hour moist cure 14 months soak	
Rose quartz	SiO_2	92	87	515	545	340	370
Smoky quartz	SiO_2	84		540		360	
Milky quartz	SiO_2	85		575		410	
Microcline	$KAlSi_3O_8$	90		105		450	
Plagioclase (Labradorite)	$NaAlSi_3O_8$ + $CaAl_2Si_2O_8$	111		210		405	
Marble	$CaCO_3$	78		90		370	
Magnesite	$MgCO_3$	155		585		730	

[1] Minerals constitute 1:4 by volume. After Thorvaldson (15).

ment in screening before a stronger pattern can develop. Thus the lag may be considerable. The long-term effect is a slow relaxation of screening and an increase in bond strength.

ACTIVATION

Heat. The previous discussions suggest that quartz, with its deep surface zone distortions, may be a bit slow at developing its potential cementation bond strength. One way to hasten readjustments in the surface zone may be by use of heat. Data by Thorvaldson (15) in Table 3 show a high strength gain of quartz mortars after autoclaving at 150 C. The same is true for mortars made with quartzite, flint, or chert, and for mortars made with highly quartz-bearing igneous rocks. In his closure to this paper Thorvaldson mentions that lime and quartz become reactive at 150 C and new reaction products are formed. With normal temperature curing the full strength apparently is not fully developed after 14 months. Thorvaldson uses these strengths as a standard for the comparison of mortars made with other minerals.

Alkali. The solubility of quartz in alkalis suggests a solution for the troublesome surface layer—namely, solution of part of the troublesome surface layer. Treatment of loess with a solution of NaOH alone gave considrable strength, probably by incorporation of the surface layer of quartz into a gel which would borrow exchangable calcium from the clay. Note that strengths came only at early ages (Fig. 4) and there is no latent strength gain as from reorganization within the mineral surface layers.

Treatment with strong alkali should improve the reactions of quartz with alkaline stabilizers such as lime or portland cement. Unpublished research shows early strengths may be raised 15 to 400 percent by use of strong alkalis. Anderson (1) attempted to correlate petrographic data with alkali reactivity of Iowa loess, and while the correlation with quartz surface area is imperfect, as shown in Figure 5, there does appear to be a relationship. The other major minerals in loess are feldspars and carbonates, discussed below. The equation for Reactivity Index is from earlier work by Handy, Davidson and Chu (8).

RELATIONS TO KINDS OF MINERALS

Feldspars and quartz. Quartz is about 50 percent ionically bonded and comparatively well screened by distortion in the surface layer. Feldspars, which are tectosilicates built on a three-dimensional framework as is quartz, contain Al^{+3} substitued for some of the Si^{+4}, the valence deficiency then being made up by Na^+, or Ca^{++}: The result is a preferred directional bonding leading to cleavage, and a net increase in the number

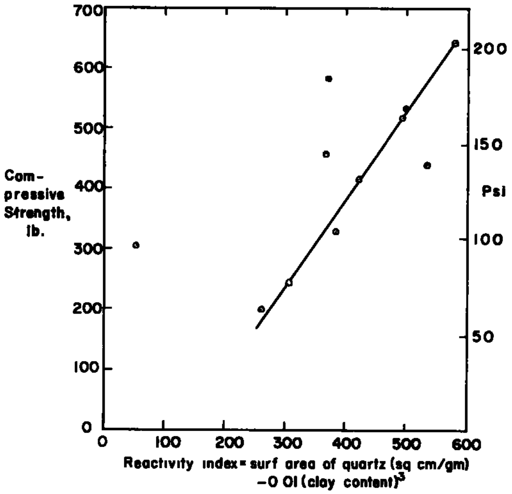


Figure 5. Compressive strength of 8N NaOH-treated 2-in. by 2-in. cylinders molded with various loess samples. Quartz surface area measured microscopically on material coarser than 5 microns.

of cations and in the ionic character of the bonding. The existence of inherent cleavage directions signifies a relatively lower surface energy along those planes; less energy is required to make a new surface. Cleavage probably takes place because of the comparatively weaker ionic bonding, especially since the nearby Si-O bonds may be strengthened by unsymmetrical polarization of the oxygen ions, giving a better Si-O covalent bond (14). It should be pointed out that in general the most stable silicate minerals are those with the most covalence.

The surface of a feldspar would be expected to contain metallic cations screened by polarization, especially since the cations, particularyly potassium, are somewhat polarizable themselves.[2] Polarization of the surface ions should offer little hindrance to cementation compared with the deeper more lasting distortions found in quartz. Such structural adjustments are probably less pronounced in feldspars because of the lower surface energy to start with, better polarization of the surface layer, and lower average valence on the cations, amounting to a deconcentration of charge. From this analysis one would expect (a) moderate early cementation bond strength, less than the potential in quartz because of the reduced but assymetrical covalent bonding, (b) no special reactivity at high temperatures, (c) a long-term strength gain unrelated to relaxation of distortion screening, and (d) no benefit from dissolution of the surface layer other than to clean the grains.

Carbonates. Calcite, $CaCO_3$, is another step in the same direction as feldspars. Calcite contains covalent CO_3 groups occurring in planes and bonded to one another by ionic O-Ca-O bonds. Because of polarization of the oxygen ions bonded to C, the Ca-O bonds are rather weak. Therefore calcite is softer and more soluble than most silicates. It also has very good cleavage. Screening at a surface would be expected to be nil. Therefore cementation bond strength should develop rapidly and show little high temperature improvement (Table 3). From the standpoint of removing screened zones there should be little advantage in chemical treatment. High long-term strengths (Table 3) are unexplained but are believed unrelated to screening.

Magnesite, $MgCO_3$, was found by Thorvaldson (15) to be peculiarly reactive on autoclaving, whether incorporated in mortar as a pure mineral or in the rock dolomite, $MgCa(CO_3)_2$. He suggests that the solubility may be a factor. $Mg(OH)_2$ is less soluble than $MgCO_3$, resulting in dissolution of the carbonate and precipitation of the hydroxide, probably as brucite. Highest strength was found after long curing (Table 3).

Clay minerals. Clay minerals and micas are platy crystals classified as phyllosilicates. Weyl (16) believes that clay flakes are so thin (7-10A) as to prevent satisfactory structural screening, and he attributes the electrical effects observed on clay to this cause. Adsorption of OH ions on the unscreened sites would serve to make the clays negative. However, isomorphous substitutions add to negativity of many clay minerals.

Cations within the clay mineral are automatically partly screened by their positions in tetrahedral and octahedral layers. Three-layer clays such as illite have predominantly O^{--} ions in their surfaces, whereas two layer minerals such as kaolinite contain both

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[2] The radius of Na^+ and Ca^{++} ions is about 1Å; that of K is about 1.33Å. Larger ions are the more polarizable. For comparison the radius of O is 1.4Å. Si^{+4}, on the other hand, has a radius of only 0.4Å. Al^{+3} has a radius of 0.5Å. Figures are from Lange, Handbook of Chemistry, 8th ed. (1952).

TABLE 4

UNCONFINED COMPRESSIVE STRENGTHS OF COMPACTED LIME-FLYASH MIXTURES. 92% FLYASH, 8% LIME (17).

Flyash	Loss on ignition	Compressive strength, psi 20 C cure 7 days	 45 days	 60 C cure 7 days	 45 days
No. 10	3%	240	2250	2250	2700
11	28%	40	190	420	570
12	10%	50	950	1200	1550
15	17%	120	900	1100	1250

O^{--} and OH^-. The average bond type within the crystal approximates that of quartz of feldspars[3], but this may be of little direct importance because of the variety of bonds occurring. The exceptionally good cleavage representing weak surface forces would be a barrier to bonding, more so than in feldspars and calcite, and there would be no surface adjustments to give slow improvement. This disregards hydration and the flocculation by excess calcium from the cement. Flocculation takes place quickly but is a comparatively weak bond. Probably more important are long-term chemical changes within the clay minerals brought on as a result of the change in the environment. Clay minerals are uniquely susceptible to such modification.

Amorphous materials. Glasses deserve close attention because of their usefulness as pozzolans. Natural glasses and flyash vary considerably in composition, but are essentially mixtures of Al_2O_3, SiO_2, and Fe_2O_3, plus lesser amounts of other oxides. The bonding is therefore mainly ionic and probably variable according to localized coordination conditions. As already discussed, screening probably occurs but is imperfect because of randomness of the inner structure. However, one would expect the reactivity of pozzolans to be particularly enhanced by alkalis or by heat because this would expose disorganized glass. The effect of heat on curing of flyash, an artificial pozzolan, is shown in Table 4. Benefits would be expected to be greatest in pozzolans highest in silica, as the higher valence on silicon means fewer cations to be screened than if the cations are Al^{+3} or Fe^{+3}. In the case of flyash, the composition varies so much from grain to grain that almost any flyash should be benefited—each grain

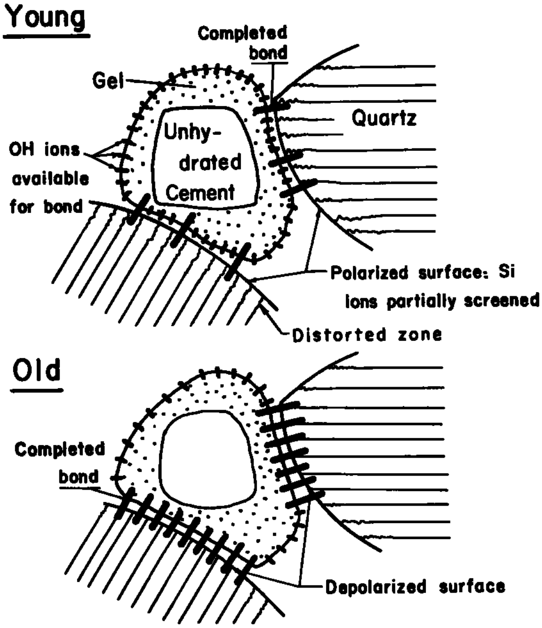

Figure 6. Diagrammetic representation of a theory of cementation. Early cementation bonding is weak due to distortion polarization of the quartz surface. However, the early chemical bonding causes distortion to slowly disappear. This in turn allows even greater chemical bonding.

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[3] Using Pauling's relationship and electronegativities from Table 1, Orthoclase, $KAlSi_3O_8$, gives $X_B = (0.8 + 1.5 + 3 \times 1.8)\ 1/5 = 1.54$. Kaolinite, $Al_2O_3 \cdot SiO_2 \cdot H_2O$ gives $X_B = (2 \times 1.5 + 1.8 + 2 \times 2.1)\ 1/5 = 1.8$. Muscovite, $K_2O \cdot 3Al_2O_3 \cdot 6SiO_2 \cdot 2H_2O$ gives $X_B = (2 \times 0.8 + 6 \times 1.5 + 6 \times 1.8 + 4 \times 2.1)\ 1/18 = 1.66$.

of flyash represents the composition of a parent mineral grain in coal; there is very little combination and averaging of the product during burning.

SUMMARY

A hypothesis is advance to explain long-term strength gains in soil-cement by readjustments within the mineral surface, tending to improve chemical bonding. The hypothesis is extended to pozzolans and representative minerals in soils and in concrete, and offers an explanation of activation of silica by heat and by alkalis.

ACKNOWLEDGMENT

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REFERENCES

1. Anderson, Reuben L., Jr., "Pozzolanic Reactivity of Loess." Unpublished M.S. thesis, Iowa State College Library, Ames (1956).
2. Bogue, R. H., and Lerch, William, "Hydration of Portland Cement Compounds." Ind. and Engr. Chem. 26: 837-847 (1934).
3. Coulson, Charles Alfred, "Valence." Oxford University Press, London (1952).
4. Devore, George W., "Surface Chemistry as a Chemical Control on Mineral Association." Jour. Geol. 64: 31-55 (1956).
5. Eitel, Wilhelm, "The Physical Chemistry of the Silicates." University of Chicago Press, Chicago, Ill. (1954).
6. Gruner, J. W., "Silicates and Their Reaction Energies." American Mineralogist 35: 137-148 (1950).
7. Handy, Richard L., "Stabilization of Iowa Loess with Portland Cement." Unpublished Ph.D. thesis, Iowa State College Library, Ames (1956).
8. Handy, R. L., Davidson, D. T., and Chu, T. Y., "Effect of Petrographic Variations of Southwestern Iowa Loess on Stabilization With Portland Cement." HRB Bull. 98: 1-20 (1955).
9. Jackson, M. L., Tyler, S. A., Willis, A. L., Bourbeau, G. A., and Pennington, R. P., "Weathering Sequence of Clay-Size Minerals in Soils and Sediments." Jour. Phys. and Coll. Chem. 52: 1237-1260 (1948).
10. Munger, Elmer C., "Bond Between Portland Cement Paste and Aggregate." Unpublished Ph.D. thesis, Iowa State College Library, Ames (1957).
11. Munger, Elmer C., Discussion to "Effect of Chemical Nature of Aggregate on Strength of Steam-Cured Portland Cement Mortars." by T. Thorvaldson, Proc. Am. Conc. Inst. 52: 1405-1408 (1956).
12. Pauling, Linus, "The Nature of the Chemical Bond." 2nd ed. Cornell University Press, New York (1942).
13. Pike, Robert G., Hubbard, Donald, and Insley, Herbert, "Mechanisms of Alkali-Aggregate Reaction." Proc. Am. Conc. Inst. 52: 13-34 (1955).
14. Ramberg, Hans, "Relative Stabilities of Some Simple Silicates as Related to the Polarization of the Oxygen Ions." Am. Min. 39: 256-271 (1954).
15. Thorvaldson, T., "Effect of Chemical Nature of Aggregate on Strength of Steam-Cured Portland Cement Mortars." Proc. Am. Conc. Inst. 52: 771-780, 1403-1413 (1955-56).
16. Weyl, W. A., "Wetting of Solids as Influenced by the Polarizability of Surface Ions." In Gomer, Robert and Smith, Eds., Structure and Properties of Solid Surfaces. pp. 147-181, The University of Chicago Press (1953).
17. Davidson, D. T., Sheeler, J. B., and Delbridge, N. G., Jr., "Reactivity of Four Types of Flyash with Lime." Presented at 37th Annual Meeting HRB, (Jan. 1958).

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