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CONTENTS

SOME FACTORS AFFECTING THE DURABILITY OF LIME-FLY ASH-AGGREGATE MIXTURES

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Lime-fly ash-aggregate (LFA) mixtures have proved to be effective and economical paving materials. In certain instances, however, these materials show a tendency to deteriorate slowly with time under severe service conditions. The objective of this study was to evaluate some of the important factors believed to influence the durability of LFA mixtures and to determine whether control of these variables can produce a more durable product. Specifically, the effect of the following parameters on the durability of LFA mixes was studied: aggregate gradation, lime plus fly ash content, ratio of lime plus fly ash to aggregate fines, curing time, fly ash content, and saturation. Strength and durability of the various mixes were evaluated by using standard procedures outlined in ASTM C 593-69. Durability was determined by the weight loss exhibited by the specimens during 12 standard freeze-thaw cycles. Results indicate that (a) for maximum durability the mix should have sufficient fines (material passing No. 4 sieve) to float the coarser aggregate particles, (b) the ratio of lime plus fly ash to fine aggregate is optimum in the range of 1:4 to 1:7, (c) durability generally increases with increased curing, (d) most mixes used in current practice have insufficient fly ash for optimum mix proportions, and (e) the effect of saturation is dependent on the freezing rate and freezing conditions. The results provide background information for design of LFA mixes to give optimum performance.

•LIME-fly ash-aggregate (LFA) mixtures have proved to be very effective and economical materials for use in base and subbase layers of pavement systems. Although LFA mixtures are relatively new, their use has increased significantly in the last several years, and with it, so has their record of field performance. From this record, it has been learned that perhaps the most significant property of this material in terms of pavement performance is durability. That is, when pavements with LFA mixtures have been exposed to the harmful effects of loading and environment, some of them have shown a typical time-related decrease in serviceability, which requires periodic maintenance to maintain the desired level of service. Through a better understanding of the problem of durability, it may be possible to build LFA pavements with a longer life expectancy and a lower maintenance requirement. This study was undertaken to investigate some of the factors that affect the durability of lime-fly ash-aggregate mixtures. More specifically, the effects of six major factors on durability were studied:

1. Aggregate gradation,
2. Lime plus fly ash content,
3. Ratio of lime plus fly ash to total fines,
4. Increased curing time,
5. Fly ash content, and
6. Saturation.

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Results from this study give insights into the LFA material durability characteristics that are influenced by parameters that can be controlled through proper mix design.

PREPARATION OF TEST SPECIMENS

All testing was done by using the standard $\frac{1}{30}$-ft^3 (940-cm^3) Proctor size specimens and was carried out in accordance with appropriate ASTM specifications.

Materials

The lime was monohydrated dolomitic lime. The fly ash was a conditioned fly ash obtained from a stockpile near Romeoville, Illinois, commonly referred to as the Will County plant. The coal burned at the time this fly ash was produced was mined in Illinois. Two aggregates were used in this study, a well-graded gravel and a well-graded crushed stone.

Mixture Design and Preparation

Three mixture proportions were chosen for use in this study. The percentages by weight of lime, fly ash, and aggregate were 2½-10-87½, 3-12-85, and 3½-14-82½. Except in one study, a lime to fly ash ratio of 1:4 was maintained. These proportions are representative of mix proportions currently in use in field applications using similar types and gradations of aggregates.

For each test a set of five specimens was prepared in two batches to minimize moisture loss. The dry materials were charged into a Lancaster mixer and thoroughly mixed before water was added to bring the mix to optimum moisture content.

Compaction

The specimens were compacted in accordance with ASTM C 593. Briefly, this specification requires that compaction be carried out at optimum moisture content by using a standard $\frac{1}{30}$-ft^3 (940-cm^3) mold and compacting the material in three layers with 25 blows per layer of a 10-lb (4.5-kg) hammer falling through an 18-in. (45-cm) drop.

Curing

All specimens were cured at 100 F (38 C) for 7 days before testing was initiated except for those specimens used to evaluate the effect of additional curing on freeze-thaw durability. All specimens were sealed in plastic bags to maintain the moisture at the mixing water content throughout the curing period.

TEST PROCEDURE

Strength and durability tests were conducted in accordance with ASTM C 593. Of each set of five specimens, three were tested for strength and two for durability. The standard durability test consisted of 12 freeze-thaw cycles ranging from -10 to 70 F (-23 to 21 C) with a 24-hour freeze cycle and a 23-hour thaw cycle. All specimens were brushed in the prescribed manner in accordance with ASTM C 593, and the weight loss for each cycle was determined.

In the study phase to determine the effect of saturation on durability, tests were performed by using the freeze-thaw durability testing unit developed by Dempsey (1).

This unit was designed to simulate freeze-thaw temperature conditions and mechanisms similar to those that exist in in-service pavement systems. This testing chamber is programmed to subject specimens to uniaxial temperature gradients similar to those measured in in-service pavements. The durability of stabilized materials was evaluated by determining the compressive strength loss after five freeze-thaw cycles in the test unit.

ANALYSIS AND DISCUSSION OF RESULTS

Aggregate Gradation

All aggregates were split on a No. 4 sieve, and the fine and coarse fractions were reblended in various ratios so that the fine (minus No. 4 sieve) component ranged from 25 to 100 percent of the total aggregate fraction. For each mix design, one set of five specimens was made at each fines content and evaluated for strength and durability.

The gradation of the aggregate was found to have a very significant effect on the density, strength, and durability of the mixes tested. For the gravel aggregate, there seemed to be a single optimum gradation for all three properties (Figure 1). The crushed stone showed a similar trend (Figures 2 and 3), but the optimum fines content for maximum durability was slightly higher than that for maximum density and strength. This is believed to be related to the density of the cementing matrix. For high strength and good durability of the mixes, it is essential to achieve high density in the matrix, i.e., in the fines portion of mixes. High overall mix density does not necessarily produce the highest matrix density, which is achieved only when the coarse aggregate is slightly less than that required for maximum overall density so that the coarse particles are floating in the matrix. This allows the compactive energy to be used in densifying the cementing matrix rather than being dissipated, overcoming shear resistance between the coarse particles. The coarse aggregate in well-bonded mixes serves only as a filler, and the integrity of the mix depends primarily on the cohesion supplied by the cemented matrix.

It is believed that, because of the increased angularity in the crushed stone aggregate, a greater dispersion of the coarse fraction is required than with the more rounded particles to eliminate the coarse aggregate interlock, and hence a slightly higher fines content was necessary to permit densification of the matrix. The large gravel particles, which are more rounded particles and have a smoother surface texture, provided less frictional resistance to densification and required a relatively lower fines content to permit densification of the matrix.

The mixes using the crushed stone aggregate, in particular, showed a very rapid decrease in durability when fines content was reduced below optimum. This also is believed to have been caused by the low cementing matrix density and the higher voids content, but may have been accentuated by the severity of the durability test method employed. Vigorous brushing of the specimen after each freeze-thaw cycle seemed to be more detrimental to specimens with a lower fines content. Regardless of why, the results clearly indicate that it is better to be on the fat side (excess fines) than on the lean side of the optimum fines content. This is an important point to consider in mix design. It might be advantageous to design a mix with a slightly higher than optimum fines content, which would reduce the probability of poor pavement performance caused by normal variations in gradation and possible segregation during placement. The excess fines would shift the mix into the less sensitive range (Figure 2) where field variability has a relatively smaller effect on the performance of the finished pavement.

Lime Plus Fly Ash Content

The effect of lime plus fly ash content on mix properties was also studied.

Figure 1. Effect of aggregate gradation for 2½-10-87½ gravel aggregate mix.

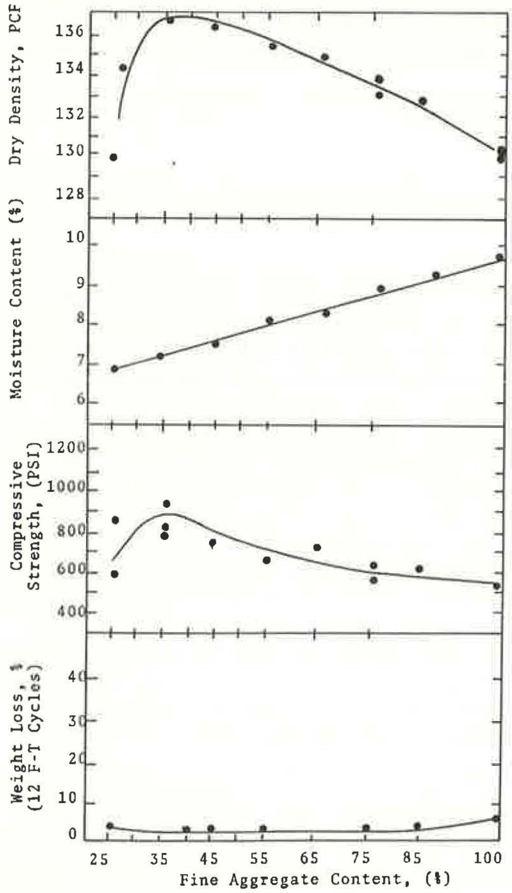
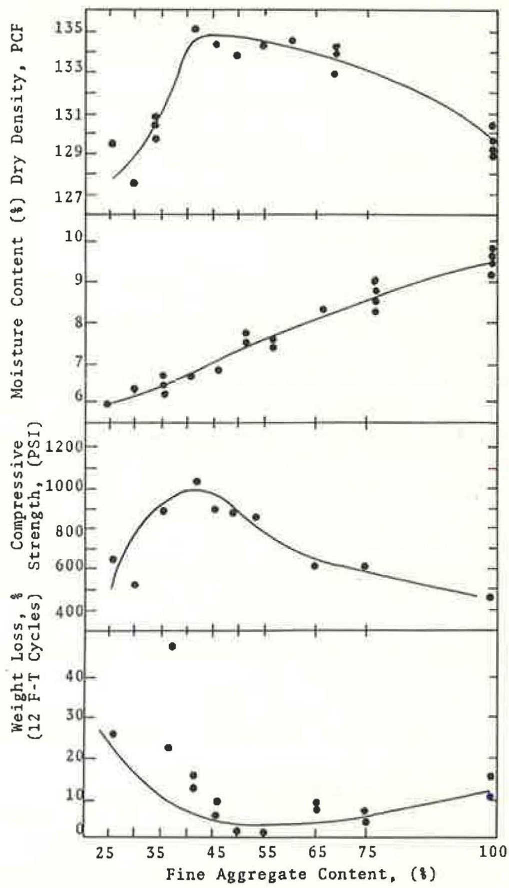


Figure 2. Effect of aggregate gradation for 2½-10-87½ crushed stone aggregate mix.



Basically, an increase in lime plus fly ash content serves to increase the percentage of cementing agents in the mix and therefore controls the quality of the cementing matrix. Its effect was most pronounced in mixes with low fines content, especially with the crushed stone aggregate (Figure 4). These mixes had a high voids content and low matrix density, which resulted in poor durability. Increasing the lime plus fly ash content helped to fill the voids and float the coarse particles and to increase the concentration of cementing products in the matrix. Small increases in the lime plus fly ash content in the mix resulted in marked improvements in durability.

The effect of the lime plus fly ash content was reduced with mixes having higher fines contents. Increased lime and fly ash provided only slight increases in durability. It is believed that these specimens already had low voids content and floating coarse particles and therefore adequate matrix density was achieved without the large lime plus fly ash contents. The effect of increasing the lime plus fly ash content diminishes as the fines content in the aggregate portion of the mix increases. These results show that the effect of the lime plus fly ash content is dependent on the aggregate gradation.

For the aggregates tested in this study, there seemed to be a range in which the mix was most sensitive to changes in the lime plus fly ash content. Decreasing the lime and fly ash content within this range resulted in large increases in weight loss, and increasing it beyond the critical range produced less dramatic results. At the higher lime plus fly ash contents, aggregate gradation had a decreasing effect on durability.

On a practical basis, this result is highly significant. Because of economical considerations, the practice has been to design mixes with the lowest lime plus fly ash content that produces adequate durability. The mix proportions used in this study (2½-10-87½, 3-12-85, and 3½-14-82½) were chosen because they were representative of the proportions most often used in field applications. Mixes with these proportions can produce durable mixes. However, the results presented show that variations in such parameters as aggregate gradation, density, and lime plus fly ash content can significantly reduce the durability of these mixes to a level where the mix is no longer acceptable. Thus, these mixes have a low factor of safety against handling errors and are sensitive to the level of control that can be maintained during mixing, compaction, and curing. By increasing the lime plus fly ash content to a level outside the sensitive range, the mix becomes durable over a wider range of conditions and a factor of safety is provided to help withstand variations in construction.

Ratio of Lime Plus Fly Ash to Total Fines

The previous discussion has shown that both the lime plus fly ash content and the aggregate gradation affect durability, and it has been suggested that some relationship exists between these parameters. An attempt was made to combine them into a single parameter, the ratio of lime plus fly ash to aggregate fines. This ratio, in effect, is a measure of the proportion of cementing agents present in the matrix. A plot of this ratio versus durability is shown in Figure 5 for all mixes tested. Each curve represents a mix with a constant lime plus fly ash content, and the percentage of aggregate fines is varied to alter the lime plus ash to aggregate fines ratio.

In Figure 5 there appears to be a relationship between this ratio and durability for a limited range. At the very low ratios, durability becomes very poor, probably because of the effect of the low fines content in the mix.

For practical purposes, then, this parameter is of limited use in the mix design process. It might serve as a guide for use in design of trial mixes by giving an indication of the range of ratios that would be expected to give good durability. The data shown suggest that mixes with a range of, say, 1:4 to 1:7 for the lime plus fly ash to aggregate fines ratio would perform well for the aggregates used in this study.

Curing Time

Studies were made to evaluate the effect of increased curing time on strength and dura-

Figure 3. Effect of aggregate gradation for 3½-14-82½ crushed stone aggregate mix.

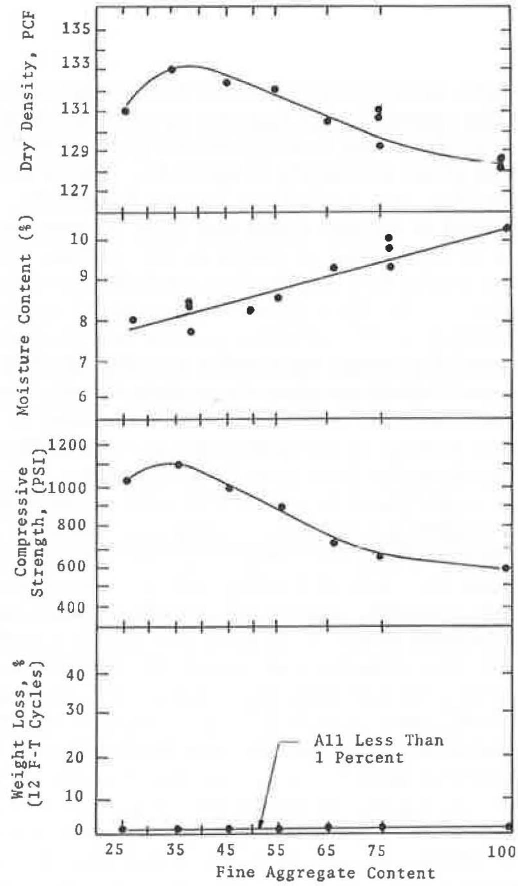
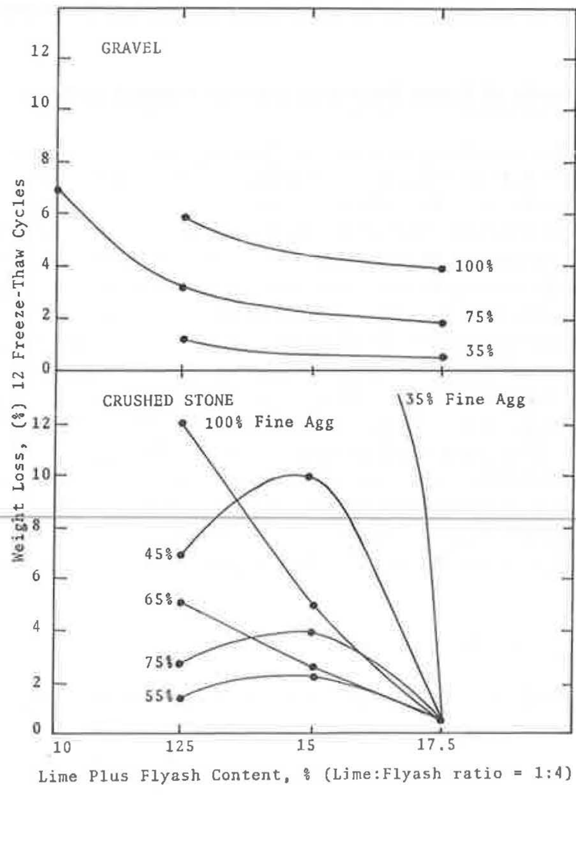


Figure 4. Effect of lime plus fly ash content.



bility. Two mixes at three fines contents (35, 75, and 100 percent fines) were chosen for this study. Specimens were cured at 100 F (38 C) for 7, 10, and 14 days before standard strength and durability were tested. The two mixes used in this study were a $2\frac{1}{2}$-10-$87\frac{1}{2}$ mix, which showed marginal durability after a standard 7-day cure, and a $3\frac{1}{2}$-14-$82\frac{1}{2}$ mix, which showed excellent durability after a similar curing period.

The results from both mixes showed that the increased curing had a substantial effect on the unconfined compressive strength. The mixes with $2\frac{1}{2}$ percent lime showed an average increase in strength of 38 percent as the curing time was increased from 7 to 14 days at 100 F (38 C), and the mixes with $3\frac{1}{2}$ percent lime showed an average increase in strength of 66 percent with the same increase in curing.

Increased curing had an important effect on the durability of the $2\frac{1}{2}$-10-$87\frac{1}{2}$ mix: Mix durability increased from marginal to excellent for all three aggregate gradations. The $3\frac{1}{2}$-14-$82\frac{1}{2}$ mix, which showed excellent durability after only 7 days of curing, showed little reduction in weight loss when subjected to the standard durability test after the additional curing.

These results suggest that increasing the strength of the lime-fly ash-aggregate mixture through additional curing beyond 7 days at 100 F (38 C) appears to be beneficial to mixes, especially those with low lime plus fly ash contents. For mixes with high durability (low weight loss) after 7 days, the additional curing does not substantially decrease the weight loss due to brushing. These results suggest that mixes with higher lime plus fly ash contents can be used to develop mixes with high performance for pavements built and used under unfavorable curing conditions. For late-season construction, for example, it would be possible to obtain adequate durability to withstand the critical first winter by increasing the lime plus fly ash content of many of the mixes currently used.

Increased Fly Ash Content

Evaluation of the mixes with $2\frac{1}{2}$ and 3 percent lime indicated that some of both mix formulations exhibited marginal durability characteristics. One possible cause was insufficiency of fine fly ash in the mix to react totally and quickly with the available lime. This would retard potential strength and durability development of the mix. A separate investigation was conducted in which the lime content was held constant at 3 percent and the flyash content varied from 9 to 18 percent. Specimens made by using the crushed stone aggregate at four fly ash contents were evaluated for strength and durability. The aggregate gradation was a 50-50 blend of fines and coarse aggregate, which corresponded to the optimum fines content determined in the mix design durability evaluation. It was felt that at optimum this gradation, the effect of an increased pozzolanic reaction, would not be masked by beneficial effects produced by the additional fly ash merely filling voids and increasing matrix density.

The results showed that the increased fly ash content had a significant effect on both strength and durability (Figure 6). Increasing the fly ash content from 9 to 18 percent increased the 7-day unconfined compressive strength from approximately 900 to 1,300 psi (6200 to 8900 kPa), an increase of approximately 50 percent. With the same change in fly ash content, weight loss during freeze-thaw decreased from 10 to 5 percent, a 50 percent reduction.

These results tend to confirm other results by the authors that show that many mixes currently used in practice could be improved substantially by increasing the fly ash content by 2 to 6 percent. Such an increase would allow for some minor segregation, which logically occurs during construction and provides a margin of safety for the material.

Saturation

Tests were conducted to determine the effect of saturation on the mix durability by using two radically different test procedures. First, standard durability tests were

Figure 5. Effect on durability of ratio of lime plus fly as to aggregate fines.

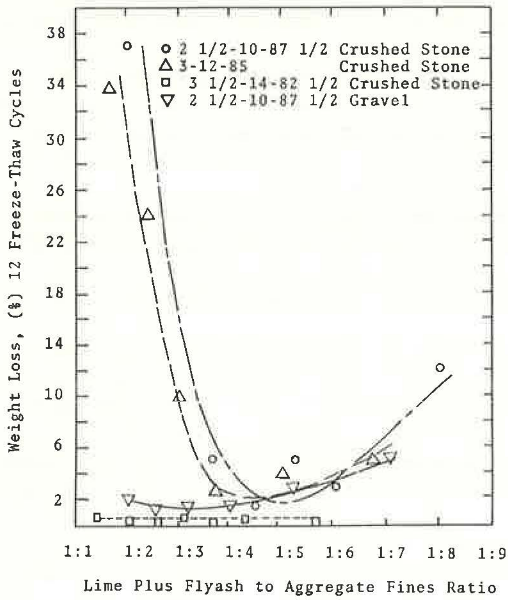


Figure 6. Effect of fly ash content.

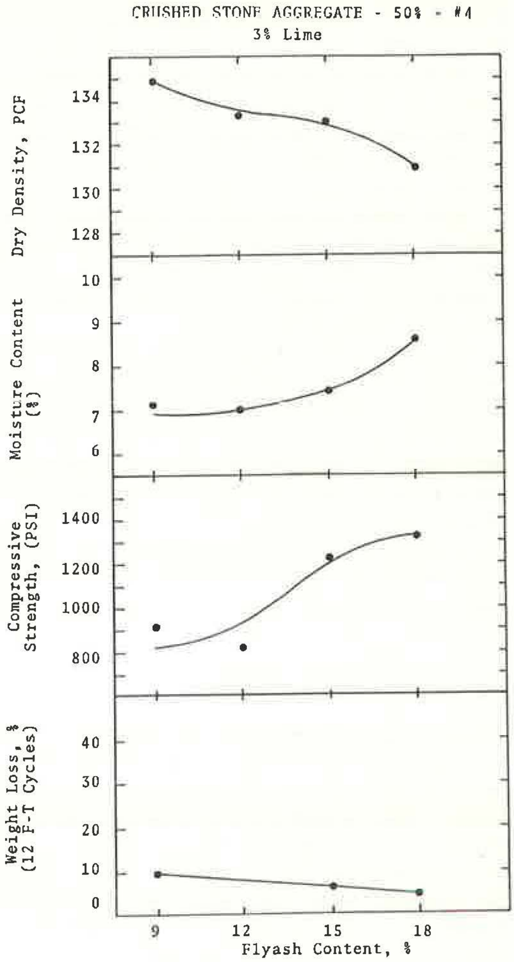


Table 1. Effect of saturation on the durability of 2½-10-87½ LFA mixtures.

Aggregate Fines (percent)	Initial Compressive Strength (psi)	Durability Weight Loss (percent)	
		Standard F-T	Saturated F-T
100	571	5.2	8.7[a]
85	634	3.2	6.0
75	649	3.0	6.2
65	717	2.0	4.4
55	674	1.8	6.8[a]
45	750	1.6	6.7[a]
35	919	1.3	9.9[a]
25	890	2.1	6.1[b]

Note: 1 psi = 6.9 kPa.

[a]Cycles during which specimens failed.
[b]Specimen failed after seven freeze-thaw cycles.

Table 2. Durability strength loss data for specimens subjected to five field-simulated freeze-thaw cycles.

Aggregate Fines (percent)	Initial Compressive Strength (psi)	Compressive Strength After Five Freeze-Thaw Cycles (psi)	
		Standard	Saturated
100	552	526	562
75	603	556	592
75	597	485	483
35	857	759	753

Note: 1 psi = 6.9 kPa.

performed on vacuum-saturated specimens and on specimens that were provided only with capillary water throughout the test. The results of this evaluation are given in Table 1. Second, durability tests were conducted on both saturated and unsaturated specimens by using the test method described by Dempsey (1), which closely simulates conditions that exist in actual pavement systems. The results of this evaluation are given in Table 2.

Comparing the results from these two tests shows conflicting data. The results given in Table 1 indicate that saturation during freeze-thaw is extremely detrimental to durability. In Table 2, the loss of strength of the saturated specimens is not significantly different from that of the unsaturated specimens, which suggests that saturation is not an important parameter. Reasons for this discrepancy become apparent when differences in the freezing mechanism of both procedures are studied.

Kennedy (2) summarized the mechanisms believed to be important in concrete scaling, and, because of similarities in matrix structure, many of the mechanisms for concrete scaling may be applicable to the breakdown of stabilized materials during freezing and thawing. One of the primary mechanisms generally believed to be responsible for freeze-thaw damage is the hydraulic pressure mechanism. Briefly, this mechanism is caused by the initiation of freezing in the pore water. As the water freezes, its expansion creates a hydraulic pressure in the remaining unfrozen pore water. This increase in pressure creates a gradient that causes a flow of pore water away from the freezing front. If this pressure is allowed to dissipate freely and quickly, the excess water will flow out of the pores and the pore will fill completely with ice without causing undue tensile stresses in the matrix surrounding the pore. This is often not the case, however, for the magnitude of the hydraulic pressure depends on several factors including (a) rate of freezing, (b) pore water flow distance, (c) matrix porosity, and (d) matrix permeability. If these factors all combine to create resistance to the flow of excess pore water, significant hydraulic pressures can build up that tend to dilate the pore structure. When the stresses induced by this hydraulic pressure exceed the tensile strength of the matrix material, freeze-thaw damage occurs, resulting in a loss of strength and durability. Such damage has apparently taken place in the saturated specimens of Table 1, which were frozen very rapidly, but not in those of Table 2 in which the freezing rate was much slower.

Specimens tested in accordance with standard durability procedures are subject to three-dimensional freezing. Freezing of the specimen begins at the outer surface and progresses inward toward the center. By the very nature of this freezing mechanism, flow of excess pore water is almost immediately cut off, causing the buildup of significant hydraulic pressures in the unfrozen pore water. These pressures can exceed the tensile strength of the matrix material and may cause significant damage of the matrix structure.

On the other hand, specimens tested in the freeze-thaw cabinet are subject to a uniaxial temperature gradient, and freezing progresses uniformly from top to bottom. This freezing mechanism does not inherently retard the flow of pore water. This water is, instead, allowed to flow dependent mainly on the rate of freezing, the length of the flow path, and the porosity and permeability of the matrix.

These results suggest that saturation is not in itself detrimental to freeze-thaw durability. The amount of damage that occurs depends on the combination of several factors, of which the degree of saturation is only one; saturated specimens can perform equally as well as unsaturated ones for a given combination of these factors. However, it must be kept in mind that the potential for freeze-thaw damage is greatly increased when high saturation prevails. The hydraulic pressure mechanism theory is valid only under nearly saturated conditions, and, if the degree of saturation is kept below a critical level, the potential for damage by this mechanism is greatly reduced. Therefore, steps should be taken to minimize the potential for saturation of LFA pavement layers in design, construction, and maintenance.

CONCLUSIONS

The following conclusions are indicated by the findings from this study. They are also supported by the authors' experience with these materials, which spans nearly 14 years of research, testing, and quality control on LFA mixes.

1. Aggregate gradation is an important factor in the durability of LFA mixes. For good durability, it is important that a high relative density be achieved in the cementitious matrix of the mix. This can be achieved only if the coarse aggregate particles are floating in the matrix. A lack of fines in coarse aggregates can be compensated for by an increase in fly ash content. It is better to have an excess of fines in the mix than to have a deficiency.
2. To achieve durability in a mix requires that the mix have adequate quantities of lime plus fly ash. The amount of lime plus fly ash required varies with aggregate gradation, but generally mixes with well-graded aggregates will require from 12 to 16 percent lime plus fly ash on a dry weight basis. Mixes with a high percentage of minus No. 4 aggregate may require a higher lime plus fly ash content to prevent diluting the matrix. Mixes with a shortage of fines may require a higher lime plus fly ash to fill the voids and float the coarse particles.
3. The ratio of lime plus fly ash to total fines appears to go through an optimum value. Ratios of lime plus fly ash to fine aggregates of from 1:4 to 1:7 appear valid for a range of mixes.
4. Increased curing time at a given temperature will result in a more durable and stronger product, particularly if the product had marginal durability under standard curing [7 days at 100 F (38 C)]. This factor is critical when LFA mix designs for late-season construction are considered.
5. Increased fly ash content has a significant effect on the strength and durability of LFA mixes. In general, even with aggregates with adequate fines, an increase in the the fly ash content results in increased strength and durability in the mix.
6. Excess water in LFA materials is detrimental to performance. The ultimate effect of saturation on the durability of LFA mixes is also influenced by other factors such as method of test, rate of freezing, and permeability. In general, however, mixes that are saturated do not perform so well as mixes that are well drained.

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EFFECT OF LIME TREATMENT ON THE RESILIENT BEHAVIOR OF FINE-GRAINED SOILS

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Lime treatment of fine-grained subgrade soils has definite potential for beneficially altering subgrade softening due to high moisture contents and freeze-thaw action. The effects of high moisture content and freeze-thaw cycles on the resilient response of a number of untreated and lime-treated soils are examined. A finite element procedure is used to evaluate the structural response of a flexible pavement on untreated and lime-treated subgrades. The analysis reveals that high moisture contents and freeze-thaw action in the subgrade have a detrimental effect on the magnitude of pavement response parameters and that lime treatment of the upper layer of the subgrade causes a substantial improvement in pavement response.

•THE BEHAVIOR and performance of pavement systems are greatly affected by subgrade support conditions. In recent years, good correlation has been found between performance and the elastic or recoverable surface deflection of flexible pavements. Because typically 60 to 70 percent of the elastic or recoverable deflection observed at the surface of a flexible pavement is accumulated in the subgrade, it is obvious that the behavior of subgrade soils subjected to repeated loads of short duration is an important consideration in pavement design.

Early work by Seed, Chan, and Lee (1) on a very limited number of soils demonstrated the various factors that affect the recoverable or resilient deformation of laboratory-prepared and laboratory-tested samples of fine-grained cohesive soils. One of the significant factors was found to be compaction moisture content or degree of saturation. Recent studies (2) conducted at the University of Illinois with a large number of soils from the Midwest, Oklahoma, Georgia, and the Carolinas also demonstrated the dramatic influence of moisture content (degree of saturation) on the resilient behavior of soils.

Moisture content during field compaction is often difficult to control or in some cases is not controlled at all (density specification only). A variation in the resilient response of a given subgrade soil can therefore be expected immediately after construction. In addition, moisture content fluctuations due to climatic factors may contribute to seasonal variations in subgrade resilient response and pavement deflections.

Recent studies by Bergan and Fredlund (3), Culley (4), Chamberlain (5), Bergan and Monismith (6), and Bergan and Culley (7) have shown that the resilient behavior of fine-grained cohesive soils is also greatly affected by cyclic freeze-thaw action. The studies revealed that, for the soils studied, a small number of freeze-thaw cycles caused a substantial increase in resilient deformation even though no moisture changes were allowed (closed system freeze-thaw).

Consequently, it is obvious that moisture variations (during and subsequent to construction) and cyclic freeze-thaw action can greatly affect the resilient behavior of the subgrade and therefore the behavior and performance of the pavement system. The pavement engineer should be able not only to predict that such variable conditions may exist but also to design a pavement system that can accommodate these variable and unpredictable conditions or alternately to design a pavement system that is fairly insensitive to changes in soil support conditions due to moisture and freeze-thaw.

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PURPOSE AND SCOPE

The purpose of this paper is to evaluate the potential of lime treatment for altering the adverse effects of high moisture contents and freeze-thaw action on the resilient response of fine-grained soils. The effects of moisture and freeze-thaw action on the resilient response of untreated and lime-treated soils and the structural behavior of pavement systems containing lime-treated soil layers are considered.

RESILIENT BEHAVIOR OF SUBGRADE SOILS

A number of factors influence the resilient response of fine-grained soils. The effect of stress level on resilient modulus is shown in Figure 1 for a typical Illinois soil. Resilient modulus is defined as applied deviator stress divided by recoverable or resilient strain. Compaction moisture and density greatly affect resilient behavior as shown in Figure 2 for the AASHO Road Test subgrade soil. The effect of small moisture content increases on the wet side of optimum is quite pronounced. Figure 3 shows the substantial decrease effected in the resilient modulus as compaction moisture content is increased (compacted density remained constant) for three typical Illinois soils.

Recent studies (3, 4, 5, 6, 7) revealed the substantial influence of closed system freeze-thaw cycling (at constant moisture content) on the resilient behavior of cohesive soils. In the study by Bergan and Monismith (6) as few as one closed system freeze-thaw cycle drastically reduced the resilient moduli values of the Regina clay soil compared to the moduli values prior to freeze-thaw cycling.

Bergan and Fredlund (3) attribute this reduction in resilient modulus to a reorientation of moisture in the soil pores and to a change in soil suction. In Chamberlain's summary of freeze-thaw weakening of fine-grained soils (5), it was indicated that localized small moisture content increases on the surfaces of "nuggets of soil particles" were sufficient to reduce cohesion between the nuggets. Chamberlain concluded, "The literature review reveals that closed system freeze-thaw will induce weakening of clay soils with or without visible ice segregation. The compressive strength and the static and repetitive moduli are affected. The rate of strength reduction due to closed system freeze-thaw decreases with an increasing number of freeze-thaw cycles."

The study by Bergan and Fredlund (3) also showed that, when a large number (10,000) of loads were applied to the Regina clay sample that had previously been subjected to the freeze-thaw cycle, the sample gained strength and displayed a resilient response similar to the sample prior to the freeze-thaw cycle (Figure 4). This sequence of testing possibly approximates the spring softening and summer strengthening phenomenon that is commonly observed for flexible pavements in regions of seasonal freeze-thaw action.

EFFECT OF SUBGRADE RESILIENCE ON PAVEMENT STRUCTURAL RESPONSE

To demonstrate the influence of subgrade resilient behavior on pavement response, we considered the flexible pavement shown in Figure 5. Two Illinois subgrade soils, one high in silt content (Fayette C) and the other high in clay content (Tama B), displaying substantially different resilient responses (Figure 6) were used under the pavement. Pertinent properties of these soils are given in Table 1.

The structural response of the flexible pavement subjected to a 9-kip (40-kN) wheel load was analyzed by using a finite element computer program that considers the stress-dependent or nonlinear response of the subgrade soil and the granular base course material.

Table 2 gives the theoretical response data obtained from the analysis of the pavement on the two subgrades. It can be noted from the data presented in this table that substantial differences in pavement structural response can be effected by differences in subgrade resilient characteristics. Note for example that the surface deflection is

Figure 1. Resilient response of typical fine-grained Illinois soil.

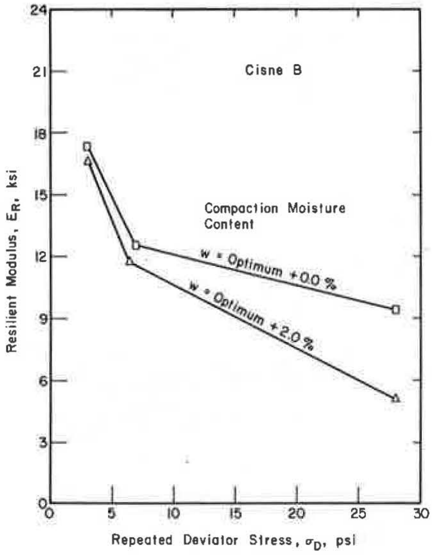


Figure 2. Effect of compaction moisture and density conditions on resilient axial strain of AASHO Road Test subgrade soil.

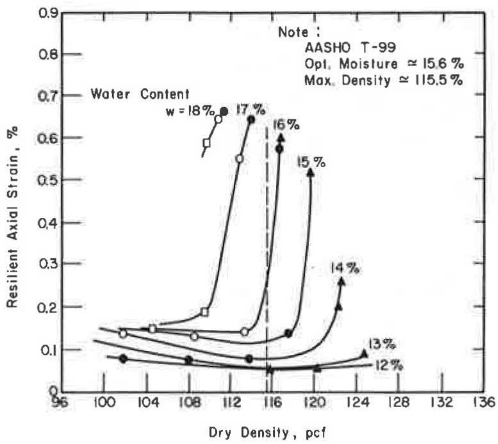


Figure 3. Effect of compaction moisture on resilient modulus.

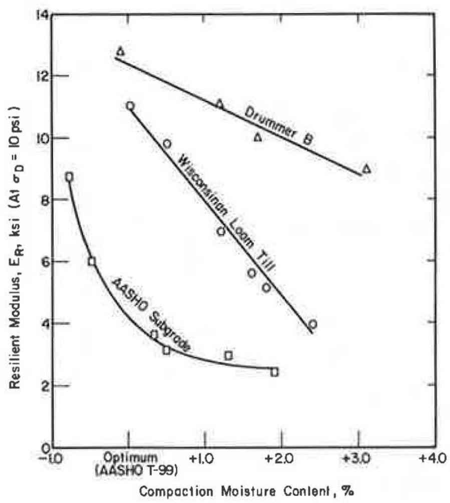


Figure 4. Resilient modulus test results before and after freeze-thaw for undisturbed Regina clay.

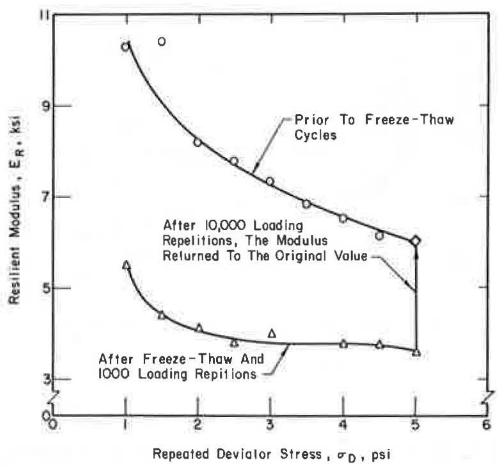


Figure 5. Dimensions and properties of flexible pavement used in study.

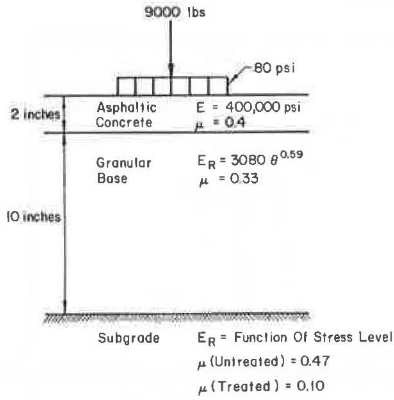


Figure 6. Resilient behavior of two subgrade soils.

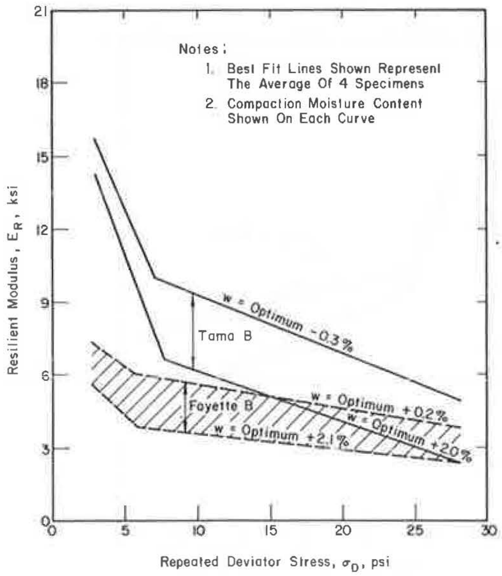


Figure 7. Effect of lime treatment and variable compaction moisture on resilient response of Flanagan B soil.

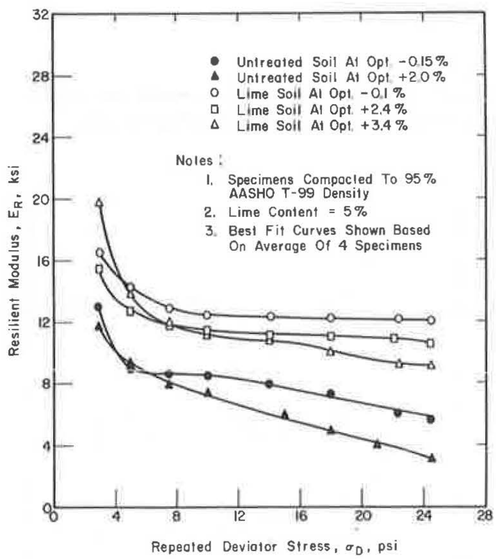


Table 1. Summary of pertinent soil properties.

Property	Fayette C	Tama B
Atterberg limits		
Liquid limit	32	46
Plasticity index	9	24
Grain size		
Percentage passing No. 200 sieve	98	100
Silt (0.002 to 0.050 mm), percent	75	67
Clay (<0.002 mm), percent	18	32
AASHO classification	A-4(9)	A-7-7(27)
Unified classification	ML	CL
Moisture-density (AASHO T-99)		
$\gamma_d$max, lb/ft^3	109.4	100.8
w_{opt}, percent	17.2	21.9
Soaked CBR	5.3	3.2
Compressive strength,[a] psi		
w = optimum	22	24
w = optimum + 2 percent	19	21

Note: 1 lb/ft^3 = 16 kg/m^3; 1 psi = 6.9 kPa.
[a]0 percent lime (95 percent AASHO T-99).

Table 2. Summary of results from analysis of pavement structural behavior.

	Surface Deflection (in.)		Radial Tensile Strain in Surface (in./in.)		Compressive Strain in Subgrade (in./in.)		Compressive Stress in Subgrade (psi)	
Subgrade Condition	Fayette C	Tama B	Fayette C	Tama B	Fayette C	Tama B	Fayette C	Tama B
Untreated								
Optimum moisture	0.060	0.041	0.00083	0.00065	0.00117	0.00076	10.9	13.1
Optimum + 2 percent	0.068	0.043	0.00087	0.00066	0.00161	0.00082	10.3	12.8
9-in. freeze-thaw layer[a]	0.073	0.055	0.00089	0.00082	0.00297	0.00186	9.9	11.6
18-in. freeze-thaw layer[a]	0.075	0.059	0.00089	0.00084	0.00271	0.00168	9.2	10.8
Lime treated								
9-in. treated layer[a]	0.050	0.040	0.00065	0.00063	0.00074	0.00037	5.3	6.9
18-in. treated layer[a]	0.045	0.039	0.00063	0.00061	0.00039	0.00021	2.8	3.8

Note: 1 in. = 2.5 cm; 1 psi = 6.9 kPa.
[a]For optimum + 2 percent compaction moisture condition.

about 60 percent greater for the pavement on the Fayette C subgrade than for the pavement on the Tama B subgrade. Similar trends are noted for the other pavement response factors. By comparing the relative magnitude of these response parameters and acknowledging that these response parameters are widely accepted as indicators of flexible pavement performance, it is obvious that substantially different potential performance would be expected from this flexible pavement on the two subgrades.

RESILIENT BEHAVIOR OF LIME-TREATED SOILS

Lime has been widely and successfully used as a stabilizing agent for fine-grained plastic soils. When lime is added to a fine-grained soil, several reactions are initiated. Cation exchange and agglomeration-flocculation reactions take place rapidly and produce immediate changes in soil plasticity, workability, and swell properties. Plasticity and swell are reduced and workability is substantially improved because of the low plasticity and the friable nature of the mixture.

Depending on the characteristics of the soil being stabilized, a soil-lime pozzolanic reaction may commence. The cementing agents formed as a result of the pozzolanic reaction increase mixture strength and durability. With some fine-grained soils, however, the pozzolanic reaction is inhibited, and extensive cementing agents are not formed. Thompson (8) has termed those soils that react with lime to produce substantial strength increases as reactive and those that display limited pozzolanic reactivity as nonreactive. For the nonreactive soils, plasticity, workability, and swell properties are beneficially altered, but strength increases are nominal. Both nonreactive and reactive soils display similar characteristics immediately after the lime and soil are mixed. It is only after the mixture is compacted and cured that substantial differences in strength are noted.

The strength properties of lime-treated soil mixtures at early ages are important in certain aspects of pavement design and construction. Neubauer and Thompson (9) reported immediate stability increases effected by lime treatment of fine-grained soils. Their study, however, was limited to a determination of static stability conditions such as shear strength, CBR, cone penetrometer value, and modulus of deformation.

From the standpoint of long-term pavement behavior and performance, it is desirable to have a knowledge and understanding of the resilient characteristics of lime-treated soil mixtures. A limited number of reactive and nonreactive soils were treated with lime, compacted into 2-in.-diameter by 4-in.-high (5- by 10-cm) specimens and immediately subjected to resilience testing (2). Complete study results may be found elsewhere (10).

Effect of Lime Treatment on Resilient Behavior

Results to date demonstrate that, without exception, the resilient response of uncured lime-treated soil is substantially different from the response of untreated soil. Figure 7 shows (a) the typical stress-dependent resilient behavior of untreated and lime-treated fine-grained soils, (b) the effect that compaction moisture content has on resilient behavior, and (c) the beneficial effect that lime treatment has on resilient behavior. The immediate effects of lime on the resilient behavior are evident. The resilient behavior is substantially improved (increased resilient modulus, reduced resilient deflection) when lime is added. Note also that, compared to the untreated soil at optimum moisture, lime treatment effects increased resilient moduli (higher resistance to repeated loads) at moisture contents as high as 3.4 percent above optimum. Other soils examined in this study (11) showed similar response trends upon lime treatment.

Freeze-Thaw Effects

As shown by previous studies (3, 4, 6, 7), a small number (as few as one) of freeze-thaw cycles apparently have an extreme softening effect on the resilient behavior of fine-grained soils. This effect was also evident in recent studies conducted at the University of Illinois (11, 12) as shown by the data in Figures 8 and 9. Based on only limited results, it appears that freeze-thaw more detrimentally affects the resilient behavior of the clayey Tama B soil than the more silty Fayette C soil. It also appears that, even though the clayey soil has substantially higher resilient moduli values after compaction than does the silty soil, a few freeze-thaw cycles cause the two soil types to display similar resilient behavior (Figures 8 and 9).

Shown in Figures 10 and 11 are the effects of additional repeated loading and 28 days of cure on the resilient behavior after initial freeze-thaw cycling. The 10,000 additional loadings appear to increase the resilient moduli of the silty soil but appear to have a minimal effect on the moduli of the clayey soil. The 28-day cure after the freeze-thaw cycling of the clayey soil also appears to have a minimal strengthening effect on resilient behavior (Figure 11).

When these two soils were treated with 5 percent commercially available high calcium hydrated lime, the detrimental effects of freeze-thaw cycling on resilient behavior appear to be minimized or eliminated. As shown in Figures 8 and 9 it can be seen that even as many as 10 freeze-thaw cycles do not significantly affect the resilient behavior. The resilient moduli of the untreated Fayette C and Tama B samples range from 3 to 6 ksi (20 to 41 MPa) after freeze-thaw cycling depending on the applied axial stress. However, when the soils are treated with lime (no cure), the resilient moduli range from 14 to 20 ksi (96 to 138 MPa) after 10 freeze-thaw cycles, showing the remarkable strengthening and desensitizing effected by the lime treatment. Note also that the resilient responses of these two lime-treated soils are very similar, although the pretreatment resilient responses were quite different.

Curing Effects

The Fayette C soil (which is considered lime reactive) was allowed to cure 20 days at 73 F (23 C) prior to freeze-thaw cycling. The resilient behavior was slightly improved compared to the uncured behavior (Figure 8). The range of resilient moduli is 18 to 24 ksi (124 to 165 MPa) depending on the magnitude of repeated deviator stress.

No additional strengthening is normally effected by curing for soils that are not lime reactive; therefore, the immediate and cured resilient responses of such materials are similar.

RESPONSE OF PAVEMENT SYSTEMS CONTAINING LIME-TREATED SOIL LAYERS

Based on previous discussion, the following facts are obvious:

1. High moisture contents and freeze-thaw action have a detrimental effect on the resilient behavior of fine-grained subgrade soils;
2. Lime treatment of fine-grained soils substantially improves the resilient moduli of the soils and reduces or eliminates the detrimental effects of freeze-thaw action; and
3. The resilient behavior of a subgrade has a profound effect on the behavior and potential performance of the flexible pavement examined.

The purpose of the following discussion is to evaluate the relative effect of a lime-treated layer of subgrade on the structural behavior and potential performance of typical flexible pavements.

The flexible pavement shown in Figure 5 was placed on the two subgrade soils, and the pavement structural behavior under a 9-kip (40-kN) wheel load was examined by

Figure 8. Effect of lime treatment, freeze-thaw, and curing on resilient response of Fayette C soil.

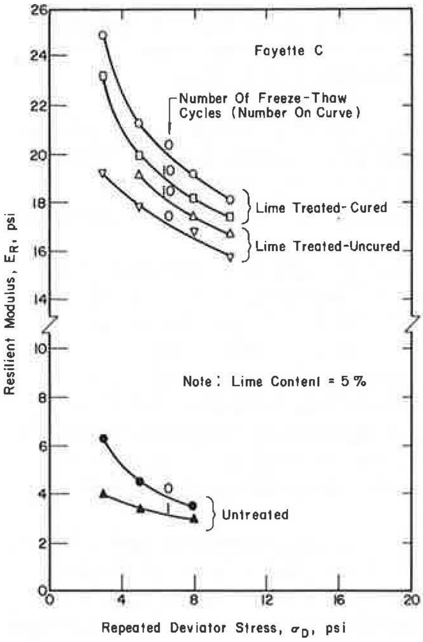


Figure 9. Effect of lime treatment, freeze-thaw, and curing on resilient response of Tama B soil.

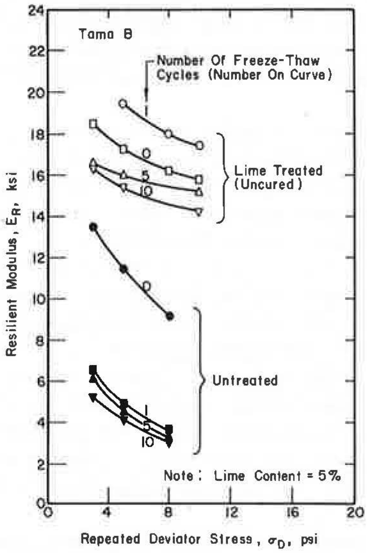


Figure 10. Effect of freeze-thaw and additional loading on resilient response of untreated Fayette C soil.

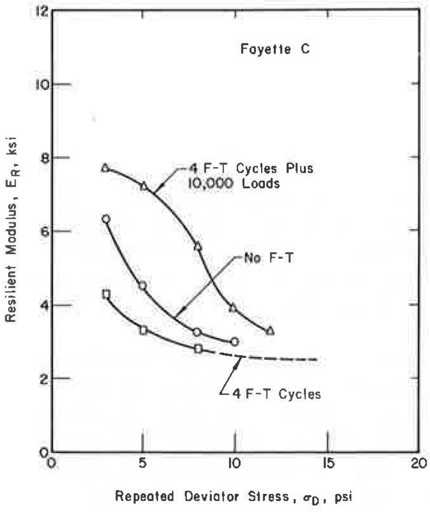
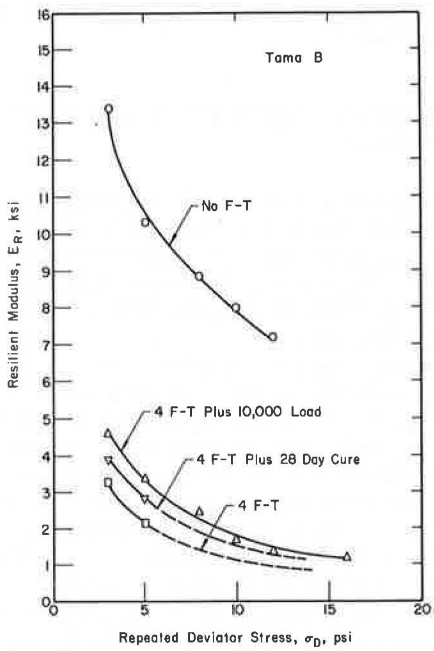


Figure 11. Effect of freeze-thaw, additional loading, and additional curing on resilient response of untreated Tama B soil.



using the finite element method previously discussed. Four variables were considered: soil type (silty and clayey), compaction moisture content (optimum and optimum +2 percent), thickness of subgrade layer subjected to the detrimental effect of freeze-thaw [9 and 18 in. (23 and 46 cm)], and thickness of lime-treated subgrade layer (9 and 18 in.).

Table 2 gives structural response data from the finite element computer analysis. By analyzing and comparing the relative magnitudes of the various response factors for the two subgrade soils, the following general observations can be made.

1. A moisture content increase from optimum to optimum +2 percent (constant dry density) leads to increases in surface deflection, surface layer radial tensile strain, compressive subgrade strain, and compressive subgrade stress. The greatest moisture-induced increases noted in the pavement response parameters are associated with the pavement on the Fayette C soil.
2. Subjecting a 9- or 18-in.-thick (23- or 46-cm) layer of subgrade soil to one or more freeze-thaw cycles, in general, causes a substantial increase in surface deflection, radial tensile strain, and compressive subgrade strain for the pavement on the clayey and silty subgrade soils compared to the condition of no freeze-thaw.
3. Treating the subgrade layer subjected to freeze-thaw with lime effects substantial reduction in the magnitude of the pavement response parameters examined. Reductions of (a) approximately 30 percent in surface deflection, (b) approximately 30 percent in radial tensile strain, (c) approximately 75 to 80 percent in compressive subgrade strain, and (d) approximately 50 to 70 percent in compressive subgrade stress are noted.
4. For the two thicknesses of lime-treated layer used in the analysis (9 and 18 in.), which coincide with the subgrade layer thickness affected by freeze-thaw, thickness appears to have only a slight effect on surface deflection and radial tensile strain but a rather substantial beneficial effect on compressive strain and stress in the untreated subgrade.

The data given in Table 2 also indicate that lime treatment of a 9- or 18-in.-thick (23- or 46-cm) layer of the subgrades that are 2 percent above optimum results in an improved pavement response compared to the pavement with a subgrade compacted at optimum moisture conditions. Thus, based on these data, lime treatment can be used to help eliminate poor subgrade resilient behavior associated with high moisture content subgrades.

DISCUSSION OF RESULTS AND SIGNIFICANCE OF FINDINGS

The phenomenon of subgrade softening or spring breakup is a very real problem associated with flexible pavements located in areas of seasonal frost. At the AASHO Road Test, substantial increases in surface deflection were noted following spring thaw (13). In an extensive study (14), the Canadian Good Roads Association found that spring Benkelman beam deflections for Canadian flexible pavements averaged 1.5 to 3 times the fall deflection values.

Numerous reasons including lower density and higher moisture content have been forwarded to explain the subgrade softening phenomenon that occurs during spring thaw. Based on the results presented here and previous studies (3, 4, 5, 6, 7), it appears that cyclic freeze-thaw even without changes in moisture conditions can cause detrimental alterations in flexible pavement behavior. These findings may at least partially serve to explain the observed spring softening of the AASHO Road Test flexible pavements, which occurred with no apparent subgrade moisture change.

Based on the limited results presented in this paper, it appears that treatment of the upper layer of the subgrade with lime will desensitize or protect the flexible pavement from the subgrade softening phenomenon associated either with freezing and thawing or moisture increase. For freeze-thaw protection, ideally, the subgrade should be treated to the maximum depth of frost penetration expected under the pavement.

Minimizing or eliminating the subgrade softening phenomenon through lime treatment will greatly enhance the pavement designer's capability for minimizing the uncertainties associated with future climatic factors (frost and moisture).

The following additional benefits may accrue from the use of lime treatment:

1. Possible reduction in pavement layer thicknesses and
2. Creation of a construction working table that will expedite construction operations and facilitate compaction of the overlying pavement layers.

One note of caution is warranted, however. Thompson (15) pointed out that, unless a certain level of strength is obtained with lime-treated fine-grained soils, frost heave may occur in the lime-treated layer under certain conditions. Based on limited results, a compressive strength of about 200 psi (1380 kPa) was needed to avoid frost heaving (15).

SUMMARY AND CONCLUSIONS

The potential of lime treatment for altering the adverse effects of high moisture contents and freeze-thaw action on the resilient behavior of fine-grained soils was examined. It was found that lime treatment greatly improves the resilient moduli of soils with high moisture contents and soils subjected to freeze-thaw cycling. It was also demonstrated that the inclusion of a lime-treated soil layer substantially improves the structural response and therefore the potential performance of a flexible pavement on a fine-grained subgrade that will be subjected to the softening effects of freeze-thaw action or moisture increase.

Other benefits may also accrue from lime treatment of the upper subgrade layer including possible reduced layer thicknesses and expedience and facilitation of construction operations.

Further research is needed, however, on the probable occurrence of frost heave in the lime-treated layer, especially for lower strength, nonreactive soils.

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CURING AND TENSILE STRENGTH CHARACTERISTICS OF AGGREGATE-LIME-POZZOLAN

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The tensile strength of aggregate-lime-pozzolan was found to be a very good indicator of the resistance of the material to freeze-thaw action. A tensile strength of approximately 68 psi (469 kPa) or greater must be attained for the material to have a reasonable chance to withstand freeze-thaw action as exhibited during the freezing season in Pennsylvania. The rate of development of tensile strength of a particular aggregate-lime-pozzolan mix is primarily a function of curing temperature. The higher the curing temperature is, the greater is the rate of gain in strength for the same amount of heat energy input. At temperatures below 50 F (10 C), no appreciable gain in tensile strength is achieved under field conditions.

•FOR the last 3 years, the Bureau of Materials, Testing and Research, Pennsylvania Department of Transportation (DOT), has been conducting extensive research on the characteristics of aggregate-lime-pozzolan material toward the following objectives:

1. To investigate the feasibility of tensile strength testing as a replacement for the standard freeze-thaw and unconfined compression tests;
2. To investigate the influence of selected variables (curing time and temperature, moisture conditions, type and amount of aggregate, amount of lime, amount of fly ash, density, and molded moisture content) on the final stabilized product; and
3. To evaluate the development of compressive and tensile strength in the field as a function of time, temperature, density, and moisture content and to correlate the results with the results obtained from laboratory tests.

When this project was initiated, it was determined impractical to study every aggregate, lime, or fly ash type. Therefore, one type of each was chosen for the initial experiment. The aggregate was a limestone with a specific gravity of 2.78 and 0.24 percent absorption. The gradation of the aggregate is shown in Figure 1. The physical properties of the fly ash and lime are as follows:

Property	Lime	Fly Ash
Percentage passing		
No. 100 sieve	96.4	97.7
No. 200 sieve	88.2	92.2
No. 325 sieve	80.2	81.6
Specific gravity		2.46
Loss on ignition	18.0	

Publication of this paper sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

The chemical properties of the lime and fly ash in percentage weight are as follows:

Chemical	Lime	Fly Ash
SiO_2	Trace	45.3
Fe_2O_3	2.0	15.6
Al_2O_3		24.6
CaO	47.4	4.2
MgO	32.6	1.3
C	1.3	2.4

Even with these limitations, the scope was still much too large based on the many mix designs that were possible. Thus, several mix designs were investigated in a preliminary study. From this study, the highest strengths were achieved with 3 percent lime, 15 percent fly ash, and 82 percent limestone aggregate. For this reason and because this mix design is very typical of most designs for aggregate-lime-pozzolan in Pennsylvania, it was chosen as the master mix design to be used for this research.

TENSILE STRENGTH AS RELATED TO FREEZE-THAW CHARACTERISTICS

Road bases stabilized with lime and fly ash admixtures may not gain sufficient strength in 7 or 28 days to satisfactorily carry heavy traffic or withstand repeated freeze-thaw cycles (1). Davidson, Mateos, and Katti (1) proposed that, for adequate freeze-thaw resistance, aggregate-lime-pozzolan bases may need a strength of 300 to 500 psi (2070 to 3450 kPa) in compression, depending on material type stabilized, thickness of the bituminous surfacing, and severity of the climate. The authors believe that the failure of aggregate-lime-pozzolan material in the field due to freeze-thaw action or instability can be related to tensile strength insufficient to sustain the induced tensile strain produced by freeze-thaw action. Thus, the tensile properties should be the prime consideration in a laboratory evaluation of the material.

Figures 2 and 3 show the relationship between tensile strength and freeze-thaw cycles. Each point on the curves in Figures 2 and 3 represents the average strength of three specimens. Each curve represents a group of samples that were placed in the freeze-thaw test (AASHO T-136-70) at the same initial tensile strength.

After each specified number of freeze-thaw cycles, three samples were subjected to the double punch tensile test as described by Fang and Chen (2). The test was conducted at a strain rate of 0.2 in./min (5 mm/min). The tensile strength was calculated from the following equation:

$$\sigma_t = \frac{P}{(1.08bH - a^2)} \tag{1}$$

where

σ_t = simple tensile strength in psi (kPa),
P = applied load in lbf (N),
b = radius of specimen in in. (mm),
H = height of specimen in in. (mm), and
a = radius of piston in in. (mm).

With this procedure, the gain or loss of tensile strength was determined in relationship

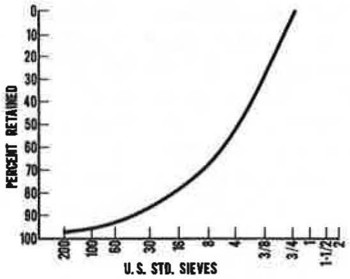


Figure 1. Gradation of aggregate used in experimental mix.

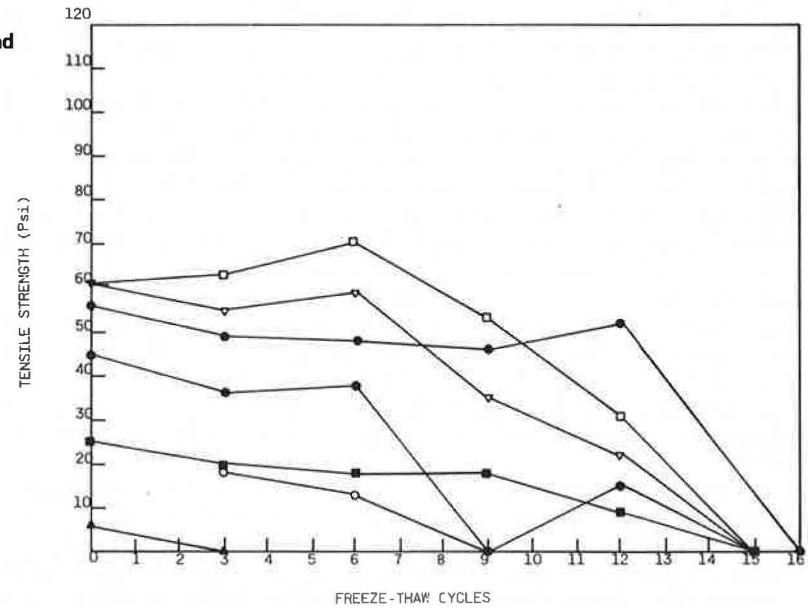


Figure 2. Relationship between tensile strength and freeze-thaw cycles for low initial tensile strength.

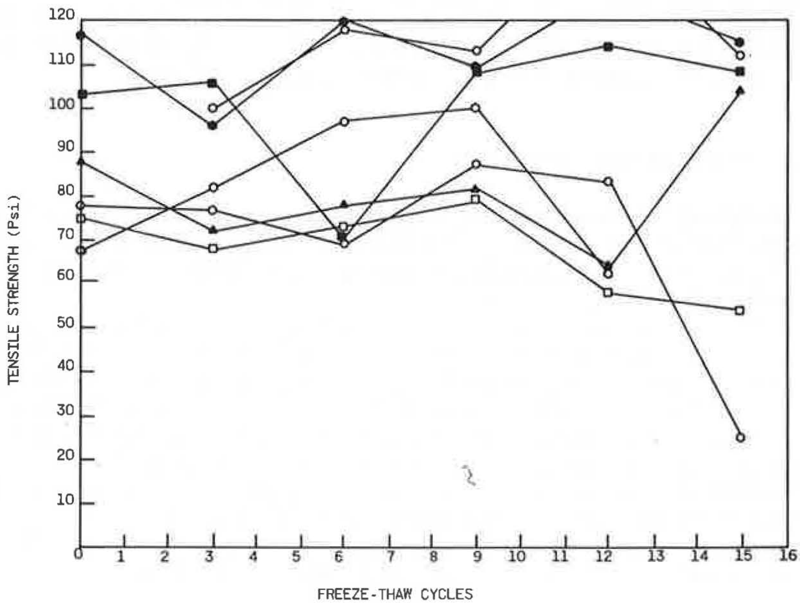


Figure 3. Relationship between tensile strength and freeze-thaw cycles for high initial tensile strength.

to the number of freeze-thaw cycles that the specimens had experienced. Table 1 gives the number of freeze-thaw test cycles that several samples withstood after they had achieved the initial tensile strength indicated (AASHO T-136-70).

Similar results were achieved from field samples cored from the shoulder of township Route 141 in Lancaster County. This material was placed between May 17 and May 27, 1971. Table 2 gives the dates when samples were cored, tensile strengths of the material when cored (average of three specimens), and number of freeze-thaw cycles (average of three specimens) that the cores withstood.

Table 3 gives the percentage loss by weight after 12 cycles of freeze-thaw when samples were placed in freeze-thaw at the initial tensile strength indicated. The type of aggregate used in developing this table was gravel instead of limestone as used in most of the research project.

Data shown in Figure 2 indicate a definite decrease in tensile strength with increasing freeze-thaw cycles. Also, note that the initial tensile strength of all the samples is below 61 psi (421 kPa) before freeze-thaw action. On the other hand, Figure 3 shows more stable tensile strengths and in some cases even an increase in tensile strength development with increasing freeze-thaw cycles. In Figure 3, the initial tensile strength of all samples is above 68 psi (469 kPa) before freeze-thaw cycling. Figures 2 and 3 show that, if a sample can withstand 15 to 18 freeze-thaw cycles, it will probably be able to withstand any reasonable number of cycles. The Pennsylvania DOT has only a few data to substantiate this conclusion. From an analysis of pavement temperatures during the last 4 years at 14 field test sites located throughout Pennsylvania, as many as 50 freeze-thaw cycles can be expected to occur at the top of the base. As many as 24 cycles can be expected in the base. A higher number of cycles occur in less severe winters. Thus, from the relatively high number and variability of freeze-thaw cycles that the base undergoes, it appears that the 12-cycle freeze-thaw test (AASHO T-136-70) is not a good indicator of actual field conditions. A possible relationship appears to be if a sample can withstand 12 freeze-thaw cycles it stands a reasonable chance of resisting a much larger number of cycles without detrimental effects.

From the data presented, failure of aggregate-lime-pozzolan material can be attributed to loss in tensile strength due to freeze-thaw action when an insufficient strength level has been achieved before freeze-thaw action begins. It appears that a tensile strength of 68 psi (469 kPa) or greater must be attained for the material to have a reasonable chance to withstand freeze-thaw action as exhibited during the freezing season in Pennsylvania. To consider a lower value of tensile strength requires that many more data be accumulated in the 50 to 70-psi (345 to 480-kPa) tensile strength range. This conclusion is valid only if there is a reliable relationship between the laboratory freeze-thaw test and what actually happens in the field. To date the literature to substantiate this relationship is rather sparse.

CURING CHARACTERISTICS

To establish a tensile strength criterion as a possible replacement for the standard freeze-thaw test and also establish a construction cutoff date for late season placement of aggregate-lime-pozzolan, a thorough knowledge of the curing characteristics of the material must be acquired.

The rate of strength gain of aggregate-lime-pozzolan is considerably influenced by the temperature at which it is cured (3, 4). The importance of this variable may be readily recognized through Figure 4. (Data for the development of Figure 4 are given in Table 4.) A higher tensile strength is achieved when the same amount of heat energy is supplied at a higher temperature. Therefore, the tensile strength is dependent not only on the amount of heat energy supplied but also on the temperature at which it is furnished. Furthermore, heat energy supplied at temperatures below 50 F (10 C) makes an insignificant contribution toward tensile strength development in the field as can be seen when the 50 F curing curve is compared with the 60 F (16 C) curve in Figure 4 and is, therefore, neglected in field application.

Table 1. Laboratory comparison of freeze-thaw cycles with tensile strength.

Initial Tensile Strength (psi)	Freeze-Thaw Cycles for 14 Percent Loss by Weight	Freeze-Thaw Cycles to Complete Failure[a]
6	3	3
25	6	15
45	9	9
56	16	16
61	15	15
61	12	15
68	12	>12
75	>12	>15
78	15	>15
88	>15	>15
103	>15	>15
117	>15	>15

Note: 1 psi = 6.9 kPa.

[a]Complete failure is defined as complete loss of samples or zero strength.

Table 2. Comparison of freeze-thaw cycles with tensile strength of cores from township Route 141.

Date Cored	Tensile Strength (psi)	Freeze-Thaw Cycles to Failure
8/14/71	44	7
9/14/71	82	12
10/10/71	88	15
2/15/72	47	5

Note: 1 psi = 6.9 kPa.

Table 3. Tensile strength and freeze-thaw characteristics of gravel.

Curing Time at 100 F (days)	Tensile Strength (psi)	Percentage Loss After 12 Cycles
3	20	100
4	20	100
6	44	51
3	29	100
4	46	42
6	51	12
3	31	75
4	61	11
6	74	10
3	27	100
4	51	47
6	72	11
3	27	100
4	43	20
6	89	9
3	27	100
4	42	100
6	92	9

Note: 1 F = 1.8 C + 32; 1 psi = 6.9 kPa.

Figure 4. Relationship between curing temperature and tensile strength.

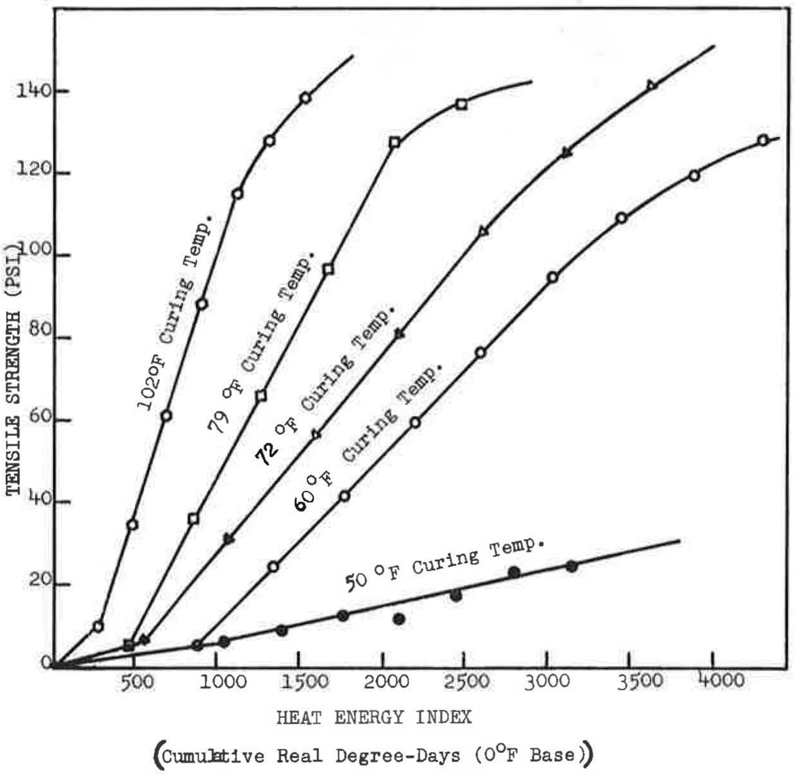
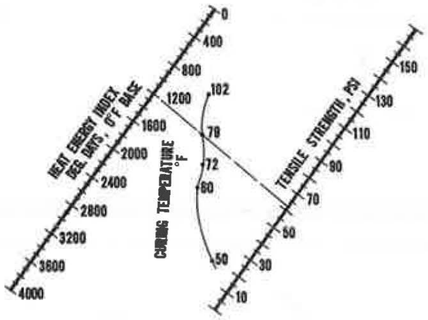


Table 4. Relationship between curing time and tensile strength for base temperature of 0 F (-18 C).

Curing Temperature (F)	Days Cured	Degree Days	Real Degree Days	σ_t (psi)
50	21	1,050	1,122	6
	28	1,400	1,472	9
	35	1,750	1,822	13
	42	2,100	2,172	12
	49	2,450	2,522	17
	56	2,800	2,872	23
	63	3,150	3,222	25
60	7	420	492	2
	14	840	912	7
	21	1,260	1,332	25
	28	1,680	1,752	42
	35	2,100	2,172	60
	42	2,520	2,592	77
	49	2,940	3,012	95
	56	3,360	3,432	110
	63	3,780	3,852	120
	70	4,200	4,272	129
	84	5,040	5,112	141
72	7	504	576	8
	14	1,008	1,080	32
72	21	1,512	1,584	57
	28	2,016	2,088	81
	35	2,520	2,592	106
	42	3,024	3,096	129
	49	3,528	3,600	142
79	5	395	467	5
	10	790	862	36
	15	1,185	1,257	66
	20	1,580	1,652	97
	25	1,975	2,047	128
	30	2,370	2,442	137
102	2	204	276	10
	4	408	480	35
	6	612	684	61
	8	816	888	88
	10	1,020	1,092	115
	12	1,224	1,296	128
	14	1,428	1,500	139

Note: Degree days = number of days x curing temperature with base temperature of 0 F. Real degree days include 1-day soak period at 72 F. 1 F = 1.8 C + 32; 1 psi = 6.9 kPa.

Figure 5. Nomogram for determining predicted tensile strength of aggregate-lime-pozzolan material.



Compaction is another variable that greatly affects the rate of strength gain in the field. The rate is significantly reduced when the material is placed below maximum density and optimum moisture content. This problem can be eliminated by strict construction control (5, 6).

Although compaction can be controlled in the field, little can be done to control the temperature. Thus, a knowledge of the curing characteristics of aggregate-lime-pozzolan is necessary for late season construction.

The authors have developed an equation and a nomogram (based on 114 samples) for predicting the tensile strength of aggregate-lime-pozzolan from curing time and temperature data. In the development of the equation for predicting tensile strength in the field, the following assumptions are made:

1. The material is compacted to maximum density and at optimum moisture content;
2. The materials and mix design are the same or similar to the ones used in this research project;
3. The 79 F (26 C) curing curve in Figure 4 approximates the curing of the material throughout the average daily temperature range greater than 75 F (24 C);
4. The 72 F (22 C) curing curve in Figure 4 approximates the curing of the material throughout the average daily temperature range of 68 to 75 F (20 to 24 C);
5. The 60 F (16 C) curing curve in Figure 4 approximates the curing of the material throughout the average daily temperature range of 55 to 67 F (13 to 19 C); and
6. The 50 F (10 C) curing curve in Figure 4 approximates the curing of the material throughout the average daily temperature range below 55 F (13 C) and will be neglected in the development of the equation.

The relationship developed by determining the equation of the curve for each curing temperature in Figure 4 is

$$\sigma_t = 8.0 + 0.041(X) + 0.049(Y) + 0.078(Z) \tag{2}$$

where

X = cumulative real degree days for the 55 to 67 F (13 to 19 C) range,
Y = cumulative real degree days for the 68 to 75 F (20 to 24 C) range, and
Z = cumulative real degree days for greater than 75 F (24 C).

It was also determined that, under field conditions, approximately 8 days were required before the curing rate reached the straight-line portion of the curves in Figure 4. In the initial 8 days of cure, an average tensile strength of 8 psi (55 kPa) was developed. Thus, in our use of equation 2, tensile strength at any time is equal to the tensile strength after 8 days of initial placement of the material (8 psi) plus the cumulative real degree days during the investigated time period and for each temperature range mentioned above multiplied by the slope of the curve in Figure 4 for that particular curing temperature range.

A nomogram was also developed (Figure 5) and is used in the following manner. Determine the heat energy index, cumulative degree days concept with 0 F base temperature (0 F was used because this gives the greater fan effect or separation of the curves in Figure 4 and it also simplifies the calculations in the conversion to cumulative degree days).

1. Determine the average daily temperature for each day during the period being investigated.
2. Sum the temperatures in the range of 55 to 67 F (13 to 19 C). Temperatures below 55 F have insignificant contributions toward tensile strength development in the field and are, therefore, neglected.
3. Sum the temperatures in the range of 68 to 75 F (20 to 24 C).

4. Sum the temperatures greater than 75 F (24 C).

Use the curing temperature scale and

5. Project a straight line from the heat energy index scale (sum of the temperatures determined in step 2 above) through 60 F (16 C) on the curing temperature scale and read the partial tensile strength,

6. Project a straight line from the heat energy index scale (sum of the temperatures determined in step 3 above) through 72 F (22 C) on the curing temperature scale and read the partial tensile strength value, and

7. Project a straight line from the heat energy index scale (sum of the temperatures determined in step 4 above) through 79 F (26 C) on the curing temperature scale and read the partial tensile strength value.

For predicting the total tensile strength development for the time period in question, add the partial tensile strength values determined in steps 5, 6, and 7.

The nomogram and equation 2 are valid only for the materials and mix design investigated in this research. Currently, industry is working on additives and different types of lime that would increase the curing rate of aggregate-lime-pozzolan to facilitate late season construction. With increased curing rates, revisions to the nomogram and equation 2 will have to be developed.

CONCLUSIONS

Based on the research work completed, the following conclusions have been drawn.

1. One of the major causes of failure in aggregate-lime-pozzolan material is the loss in tensile strength caused by freeze-thaw action.
2. A tensile strength of approximately 68 psi (469 kPa) or greater should be attained in order for the material to have a reasonable chance to withstand freeze-thaw action as exhibited during the freezing season in Pennsylvania. This tensile strength must be achieved before the first freeze of the season. This conclusion also leads to the recommendation that a construction cutoff date be established based on the probability of failure at a given tensile strength and a statistical analysis of Pennsylvania weather conditions with respect to the curing characteristics of aggregate-lime-pozzolan. For construction after the cutoff date, additives or a different type of lime in the mix would have to be used to increase the rate of gain in tensile strength development.
3. The rate of gain in tensile strength of a particular mix is primarily a function of curing temperature. The higher the curing temperature is, the greater is the gain in strength for the same amount of heat energy input.
4. At temperatures below 50 F (10 C), no appreciable gain in tensile strength is achieved under field conditions.
5. For the materials and mix design investigated in this research, equation 2 may be used to predict the tensile strength development of aggregate-lime-pozzolan in the field.
6. The feasibility of replacing the standard freeze-thaw test with a tensile strength test is excellent.

ACKNOWLEDGMENTS

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the Pennsylvania Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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DISCUSSION

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The authors have shown very useful data by using the double punch tensile strength related to the freeze-thaw action of aggregate-lime-pozzolan material. Figure 4 developed by the authors has a significant value for further material classifications based on the tensile strength. Because the double punch test was developed at Lehigh in early 1970 (2), the writer wishes to update research results and make additional comments regarding the double punch tensile test that relate to the basic procedure on stabilized soil.

The parameter (1.08) shown in equation 1 is not a constant. It depends on the specimen size and material types. For practical purposes, the following values have been recommended (9, 11):

Mold	Soil	Stabilized Materials
Proctor, 4 × 4.6 in.	1.0	1.2
CBR, 6 × 7 in.	0.8	1.0

The effect of the loading rate on the tensile strength has been studied (9, 13). These results show that there is no definite trend in tensile strength variation or deformation at failure when the loading rate varies from 0.03 to 2.0 in./min (0.7 to 51 mm/min). It was, therefore, recommended that the ASTM loading rate for the unconfined compression test be used for the double punch test. The effect of punch size is essential, and, based on experimental results, the ratios of the diameter of the specimen to the diameter of the disk (punch) of 0.2 to 0.3 are suitable for the test (2, 14). However, both theoretical (7) and laboratory studies show that the shape of the specimen does not affect the double punch tensile results. Because the double punch test is a type of penetration test on unconfined soil mass, the cracks always travel in the shortest distance from the center of the punch. The test has been extended to test bricks, masonry block, compacted and stabilized waste disposal material, and polymer-concrete block. Furthermore, the tensile test together with the unconfined compression test can be used for estimating other strength parameters, cohesion, and ϕ—the internal friction angle based on the method proposed by Fang and Hirst (10).

As previously pointed out, the conventional split tensile test measures the value of tensile strength across a predetermined failure plane, whereas the double punch test always causes failure on the weakest plane (random failure plane), resulting in a measurement of the true tensile strength of the soil (11, 12, 13). Tests have shown that, for rocks (8) and stabilized materials with high nonhomogeneous properties, results from the double punch test are lower than those obtained from the split tensile test. Because of a random failure plane, the double punch test is a very useful and sensitive method for studying the consistency characteristics and classification of soil, stabilized soils, and other construction materials.

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THE WACO PONDING PROJECT

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This report presents results of field studies conducted between 1957 and 1972 on the effectiveness of ponding and lime stabilization of clay subgrade to minimize volume change beneath portland cement concrete pavements. Potential vertical rise (PVR) was calculated to identify sections in need of ponding, and the relationship of PVR to roughness and heaving of pavement is presented. The thickness of asphaltic concrete overlay required for pavement over untreated subgrade is compared to that required for concrete pavement over lime-stabilized subgrade, some of which was ponded. Although a study of underdrains was not intended as part of this project, it became noticeable that the result of connecting perforated underdrains to ditch drop inlets was to increase heaving and overlay repair thicknesses. A method for determining desired moisture content is presented, and it correlates fairly well with moisture contents obtained from below pavement after several years.

•THIS paper discusses work begun in Waco, Texas, in 1957 to decrease the detrimental effects of heaving suffered by portland cement pavement previously placed in that area.

The term Waco Ponding Project applies to special portions of an 8.133-mile (13.08-km) project on Interstate 35 in McLennan County, Texas (Figure 1). More specifically the term applies to 18 sections (26.7 percent of the total length of project) in the southbound main lanes between Elm Mott and a point just north of West (Figure 1 and Table 1). These sections varied in length from 200 to 1,600 ft (61 to 488 m) and were distributed throughout the length of the project. They were selected as representative of some of the highest volume change conditions on the project. The project was not set up for research study, but numerous investigations were performed, including moisture movements, pavement movements, and pavement performance over a period of several years. There were two concrete highways side by side; one was 35 years old and the other was 5 years old. Both highways were rough and in need of leveling to improve serviceability.

The basic geological units encountered on this location are Upper Eagle Ford group, Austin chalk, and Lower Taylor marl member. The Lower Taylor member is the predominant unit, for approximately 75 percent of the location is over this outcrop area.

The Taylor formation is a neritic marine unit deposited near the edge of the stable Texas craton. The Lower Taylor marl is a dark gray to dark yellow clay. Fresh exposures display blocky conchoidal fracture and develop poor fissility and lighter color upon weathering. The marl is composed of silt-sized quartz, calcite fragments, phosphate nodules, hematite, and finely disseminated pyrite and pyrite nodules. The dominant clay mineral is montmorillonite.

Soils identified by soil series as established by the Soil Conservation Service are as follows: Houston black clay, Houston clay, Wilson clay, Burleson clay, Axtell fine

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*Mr. Kelly was formerly with the Texas Highway Department, Waco.

Figure 1. Location of study sections.

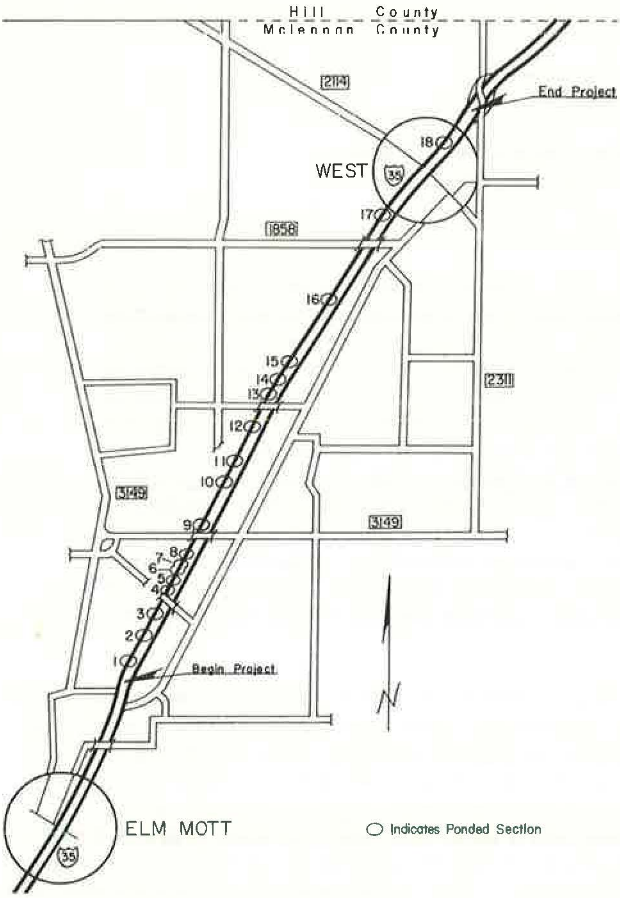


Table 1. Station limits of sections.

Section Number	Station Limits	Length (ft)	Percentage of Total Length
1	482+00 to 487+50	550	1.3
2	499+00 to 505+00	600	1.4
3	512+00 to 514+00	200	0.5
4	526+00 to 529+00	300	0.7
5	532+00 to 537+00	500	1.2
6[a]	539+00 to 545+00	600	1.4
7[a]	542+00 to 550+00	800	1.8
8	551+00 to 557+00	600	1.4
9	572+00 to 588+00	1,600	3.7
10	617+00 to 620+00	300	0.7
11	629+00 to 633+00	400	0.9
12	653+00 to 657+00	400	0.9
13	666+00 to 671+00	500	1.2
14	682+00 to 688+00	600	1.4
15	698+00 to 709+00	1,100	2.6
16	744+00 to 749+00	500	1.2
17	802+00 to 817+00	1,500	3.5
18	863+00 to 870+00	700	1.6

Note: 1 ft = 0.3 m.

[a]Overlap area.

sandy loam, Irving sandy loam and clay loam, and Austin-Eddy soils over the Austin chalk outcrop area.

Most of the soils listed are heavy black to gray residual clays with high to extremely high plasticity indexes.

HISTORY AND DETAILS OF THE PROJECT

When the southbound main lanes of I-35 between Elm Mott and West were constructed in the late 1950s, the most promising techniques for improving highway pavement performance were tried in this area where pavement performance had been very poor. The intent was to decrease the detrimental effects of heaving suffered by portland cement concrete pavements previously placed in the same area.

Of all the factors influencing volume change, addition and control of moisture appeared to be the only practicable remedial procedure to consider. Compaction control using optimum moisture on the disturbed layers was also expected to contribute favorably on fill sections. To aid in moisture retention after ponding, the lime-stabilized subgrade was extended to the width of the ponded section. These sections were ponded and stabilized with lime in 1958.

When the construction project of 1957 was conceived, the existing facility consisted of a four-lane divided highway. The southbound section was a concrete pavement completed in 1933, and the northbound pavement was a concrete section completed in 1952. The serviceability of these two sections of pavement was so low that the construction project was to include leveling and overlay of these pavements. This loss in serviceability was not the result of loss in structural capabilities but was attributed primarily to the characteristic volume change in the naturally occurring soils in the area.

Prior to this date, the Portland Cement Association had conducted a brief experiment in the area that indicated that soaking could produce the desired volume change prior to construction (1) and could eliminate the differential vertical movement that was plaguing all construction in the area. McDowell's report (2) on potential vertical rise had correlated very closely with the findings of the Portland Cement Association.

As a result of this close correlation, limited sections of the subgrade were to be ponded prior to construction of the section that was to become the southbound main lanes of the Interstate highway in the area. The plan included construction of an east frontage road with flexible base and penetration surface and the use of the existing southbound lanes as the west frontage road. The existing northbound lanes were to remain as the northbound lanes, and new southbound lanes were to be constructed between the existing surfaces.

Certain areas of the existing ground were to be ponded prior to grading, and then the area was to be graded and the surface of the grading protected by lime stabilizing of the surface of the subgrade. Because funds would not be available to pond the entire area, basic criteria were established for selecting the sites to be ponded. The initial investigation was carried on by personnel of the District Laboratory of Texas Highway Department District 9 at Waco.

Areas to be ponded were selected on the basis of potential vertical rise (PVR) in excess of 1 in. (2.5 cm). The method used to calculate the PVR was generally the same as the present procedure Tex-124-E. According to this method, extensive push-barrel sampling is necessary to determine existing moisture content and soil constants. Because not all of the locations could be or needed to be ponded, it was decided to consider the areas where proposed fill was less than 6 ft (1.8 m).

This was based on the premise that moisture and density of the fills would be controlled so as to minimize swell from the fills themselves and that the surcharge load from 6 ft (1.8 m) of fill and 24 in. (0.6 m) of pavement would be sufficient to restrain swell of most sublayers. Deep cuts were investigated after the material was removed to approximately the proposed grade line. In areas of severe swell, it was found necessary to investigate to a depth of 20 ft (6 m) because the potential swell would not load out above 16 to 18 ft (4.9 to 5.5 m). The contractor was supplied with the limits of these areas, and he then diked and ponded 18 experimental sections in accordance with the plans and specifications, for 30 days, at which time he was allowed to remove the ponds and proceed with the grading in accordance with standard Texas Highway Department procedures.

A typical cross section of both the northbound and southbound main lanes is shown in Figure 2. The proposed southbound main lane (SBML) concrete pavement contained corrugated metal contraction joints on 15-ft (4.6-m) centers and no expansion joints except at bridge ends. The portland cement concrete was designed and constructed to have a minimum 7-day flexural strength of 650 psi (4480 kPa). The SBML probably is stronger structurally than the NBML; however, because all observed distortions causing uncomfortable riding appeared to be a result of heaving, it is doubtful that the structure strength had appreciable influence on the road's performance.

The structural section of the pavement (Figure 2) called for a 6-in. (15-cm) lime-stabilized subgrade, a 5-in. (13-cm) foundation course, and 12 in. (30 cm) of non-reinforced concrete pavement.

Studies of soils with a range in plasticity indexes of 25 to 55 indicated that the triaxial strength according to test method Tex-117-E improved from class 5 to class 1 with the addition of 6 percent hydrated lime by weight. During construction, 25 lb (11 kg) of hydrated lime per square yard (0.8 m^2) or approximately 6 percent lime by weight was added to the subgrade. The PI of the lime-treated soil taken from the road varied from 6 to 21. Preliminary results of unconfined compression tests on 18-day cured specimens containing 6 percent lime varied from 62 to 281 psi (427 to 1937 kPa).

Shoulders were constructed of 6-in. (15-cm) soil cement base using 8 percent cement by volume with a 1½-in. (3.8-cm) type D asphaltic surface. The intention of the specification was to require that the average minimum unconfined compressive strength be not less than 700 psi (4830 kPa) after specimens were moist cured for a period of 7 days.

Observations of pavement performance and moisture content tests were continued over a period of several years. In 1971 the Center for Highway Research entered into a cooperative agreement to write up the data for this project. It was then necessary to find some means of describing the distortion of the original pavement slabs. Because the experimental ponded sections were scattered throughout the length of the project and because pavement performance varied between as well as within sections, it became necessary to evaluate this project thoroughly before conclusions could be formed. Thirteen years had elapsed since the pavement had been placed, so it was necessary to determine the thickness of overlay, moisture contents, and performance records as a basis for evaluating ponding. At this time the pavement had been overlayed with hot-mixed asphaltic concrete and sealed so effectively that it was difficult to conclude whether any one section was better than another. No cracks were evident, so the possibility of making a crack survey was eliminated. Profilometer measurements were made on the entire 8-mile (13-km) project, but because no previous measurements had been made they failed to reveal the past behavior of these pavements. The measurements are on file at the District Laboratory of the Texas Highway Department in Waco.

Because field bump survey observations made during a 7-year period indicated that pavement roughness developed erratically between sections, it was decided to determine the thickness of asphaltic concrete (AC) overlay at regular intervals. After the project was restaked, the thickness of AC overlay was determined at each 100-ft (30.5-m) station for all experimental sections and each opposite station in the older northbound main lanes. Detailed logs of the core thickness, which varied from nearly 2 to more than 19 in. (5 to 48 cm), were made.

COLLECTION AND ANALYSIS OF DATA

Field exploration and testing were started in May 1957. The initial investigation consisted of classification and surface mapping of soil series and pertinent geological factors. Because fills in excess of 6 ft (1.8 m) were to be excluded, the limits of these areas were established. This information, along with test data obtained at all stages of operation, was recorded on a continuous roll profile plot. Thirty-seven locations were selected for detailed testing. One hole was drilled in each location for primary data.

The necessary data obtained from each pilot hole were soil constants, gradation, moisture content, and physical description. Generally, samples were taken every 6 in. (15 cm) in depth. Some deviation from this procedure was necessary because of material changes. After tests were completed, grouping into major units could be accomplished when control factors were in agreement. At the completion of this phase of investigation, 477 samples had been tested.

Calculations of PVR values were made from the pilot hole data from each location as tests were finished. After all locations were checked for PVR, 18 of these indicated swell potential in excess of 1 in. (2.5 cm). All test data were then plotted on the profile roll, and the 18 sections proposed for ponding were established. The sections in cut areas were investigated further after completion of excavation.

The equilibrium or desired moisture content was determined for each section selected for ponding. A detailed explanation of the procedure used in the calculations of desired moisture is given elsewhere (4).

Section 1 was diked and flooded on October 10, 1957. The next section was ponded the following day. These sections were observed and all details evaluated. Full-depth moisture tests were taken after 14, 20, and 24 days of soaking (Figure 3). Test information as well as experience gained from these first sections was essential to the planning of a workable procedure for the remainder of the operation. Moisture tests during the initial ponding indicated that about 30 days' soaking on most of the sections would be adequate to reach equilibrium moisture content. Sections 5 and 7 were ponded for less time because the required change in moisture (from start of soaking to desired content) was not so great as for the average section.

There had been considerable doubt whether the truck-mounted drill would work as a self-propelled unit in ponded areas. This was accomplished with the use of removable wrap-around plated tracks on the tandem rear wheels. With this equipment it was possible to move into a ponded area immediately after the water was drained and proceed with push-barrel sampling.

An approximate time requirement for soaking was established after the first two sections; however, confirmation moisture tests were continued through the remainder of the project. Extended soaking time was continued on a random basis on several sections. Results through 1964 were used in Figure 3. Moisture tests were taken in sections 1 and 9 as late as March 1972. The latest tests (taken at the edge of the concrete pavement) were not considered to be reliable because of the depth [6 to 10 ft (1.8 to 3 m)] of the crack that had developed between the pavement and shoulder. It is very probable that the condition of this joint has had a detrimental effect on the pavement serviceability in several sections.

More than 9,000 moisture tests were taken on this project. Approximately 85 percent of these tests were made during planning and construction. All moisture and soil constant test results are on file in the Waco District Laboratory. A future project of the same length over similar materials could probably be adequately planned and controlled with fewer moisture samples, but a reduction in soil constants (477) would be questionable.

The results of moisture content tests at various depths and at various stages of the project's history for sections 1 and 9 are shown in Figures 3 and 4. The moisture content curves are the averages of many samples taken at 6-in. (15-cm) depth intervals and are believed to represent the range of values in the other sections. The LL and PI values are shown along the right edge of Figures 3 and 4. The moisture content curves shown in Figures 3 and 4 are presented in such a manner as to depict the moisture conditions before, during, and after ponding at depths below the top of the concrete pavement grade. The data shown in Figure 3 are believed to be typical of most of the sections, especially those that performed well, and the curves shown in Figure 4 are believed to be typical of a few sections that performed poorly. Many

Figure 2. Typical highway cross section.

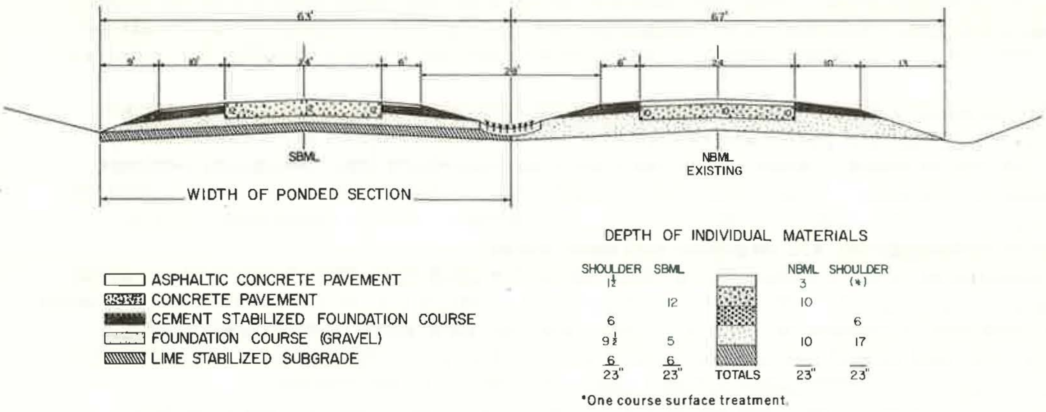


Figure 3. Depth versus moisture prior to and during ponding (section 1).

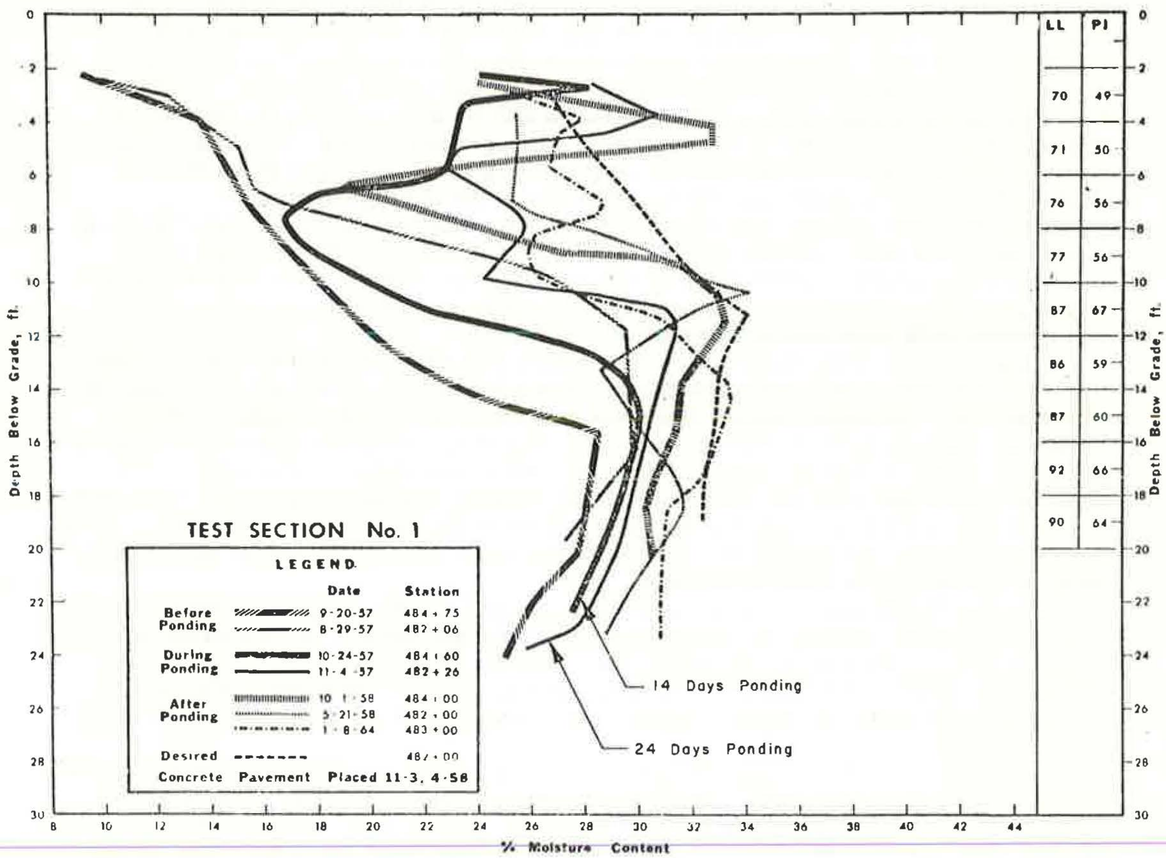


Figure 4. Depth versus moisture prior to and during ponding (section 9).

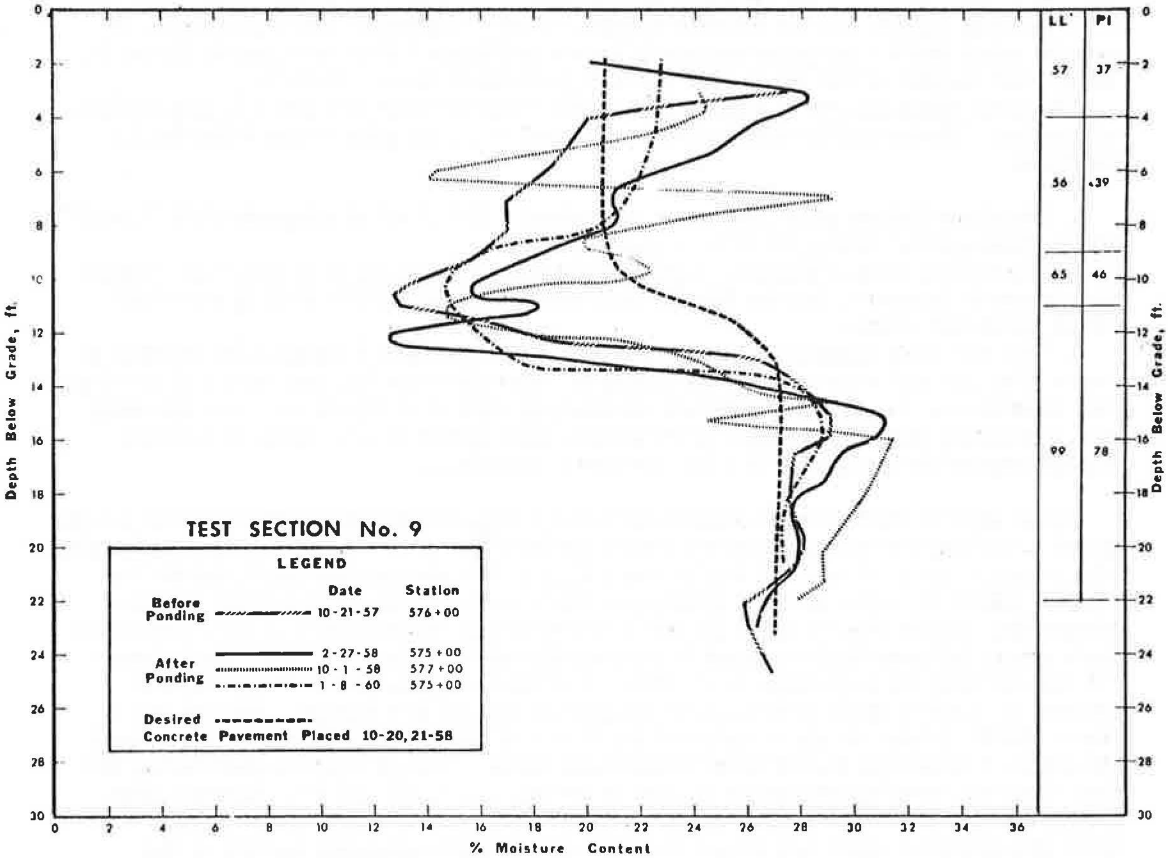


Table 2. Bumps occurring between ponded sections from 1958 to 1965.

Station Numbers	Section Numbers	Number of Bumps	Original Average PVR (in.)	Remarks
459+70.4 to 482+00	So. end to 1	4	2.0	Ponding probably would have prevented bumps.
487+50 to 499+00	1 to 2	1	No data	
505+00 to 512+00	2 to 3	0	No data	
514+00 to 526+00	3 to 4	3	1.0	Ponding probably would have prevented bumps.
529+00 to 532+00	4 to 5	1	No data	
537+00 to 539+00	5 to 6	0	No data	
550+00 to 551+00	7 to 8	0	No data	
557+00 to 572+00	8 to 9	2	0.8	Ponding probably would have prevented bumps.
588+00 to 617+00	9 to 10	4	1.4	Underdrains connected to ditch drop inlets.
620+00 to 629+00	10 to 11	1	No data	
633+00 to 653+00	11 to 12	9	3.2	Ponding probably would have prevented bumps.
657+00 to 666+00	12 to 13	1	0.9	Ponding probably would have prevented bumps.
671+00 to 682+00	13 to 14	0	0.2	
688+00 to 698+00	14 to 15	2	0.0	
709+00 to 744+00	15 to 16	2	0.8	Ponding probably would have prevented bumps.
749+00 to 802+00	16 to 17	0	1.0	
817+00 to 863+00	17 to 18	9	3.8	Ponding probably would have prevented bumps.
870+00 to 889+00	18 to No. end	4	1.9	Ponding probably would have prevented bumps.
Total		43[a]		

Note: 1 in. = 2.5 cm.

[a]38 or 88 percent could have been eliminated by ponding more extensively and/or avoiding the attachment of underdrains to ditch drop inlets.

other samples were taken in less detail in all of the other 18 sections, but for the sake of brevity they are not included in this paper. Usually the areas bounded by the after ponding curves and the desired moisture curve represent swell potential. It may be noted that these areas are much larger in Figure 4 than they are in Figure 3. This could be part of the reason for the poor performance of section 9.

The solid black curves in Figure 3 show the moisture contents after 14 and 24 days of ponding. These and the desired moisture content curve point to the following indications.

1. Moisture did not penetrate more than about 4 ft (1.2 m) of subgrade [6 ft (1.8 m) below finish grade] during 24 days of ponding.
2. During the time of ponding, moisture contents at 16 to 20-ft (4.9 to 6-m) depths also began to increase, leaving the driest areas at a depth of 5 to 10 ft (1.5 to 3 m) below pavement grade.
3. The dot-dash curve shows that moisture contents taken 7 years after ponding (6 years after paving) were slightly in excess of those shown at the conclusion of ponding. The dashed line, for desired moisture content, is located fairly close to the dot-dash line, indicating that the moisture content to be anticipated in clay soils, at various depths, can be determined with a fair degree of accuracy.

From 1958 to 1965 several observations for bumps in both the northbound and southbound main lanes were made for the entire project (Table 2). This was done by driving a passenger car at 60 mph (96 km/h) and noting on the plan profile sheet the station number where the deformations causing uncomfortable riding were located. It is interesting to note that on April 12, 1961, three bumps had occurred in the northbound main lanes, but none had occurred in the southbound main lanes. Subsequent observations for heaving were made until 1965, at which time placement of intermittent patches of overlay made it difficult to accurately record new bumps. After 3 more years (1968), it was decided to place 2 in. (5 cm) of hot-mixed AC overlay throughout the entire 8 miles (13 km) of southbound main lanes. Figure 5 shows that during the first 7 years one bump occurred in each of sections 5, 8, 17, and 18. Another four bumps occurred in section 9. The cross-hatched bars on the right side of Figure 5 show that twice as many bumps per mile occurred in the unponded portion of the southbound main lanes as in the ponded sections. Reference to the plans reveals that sections 9, 17, and 18 are the only ponded sections that have drop inlets connected to perforated underdrains. Section 9 has ditch drop inlets for large drainage areas connected to underdrains. When it rains, a head of water can back up into the underdrains, and during dry weather wide belts of soil can dry out due to evaporation. This makes for extreme fluctuations of moisture and volume change in this section of high volume change soils. It is possible that these bumps might not have occurred if underdrains had not been connected to drop inlets. In this case, there would have been four bumps in all sections or only about one-fourth as many per mile as occurred in the unponded portion. One bump each occurred in sections 17 and 18. These sections were in small drainage areas such as those at underpasses, and the drop inlets were at curbs. If all bumps from ponded sections that have drop inlets connected to underdrains are ruled out, there would be only two bumps left in all sections, and the unponded portion could be said to have between seven and eight times as many bumps per mile as the ponded sections. The reader's interpretation of these findings will depend a great deal on his background and attitude, but the most conservative reader must conclude that ponding was beneficial. The authors believe that it would have been possible to construct large portions of this project so that they would have remained relatively free from heaving.

Because PVR has been used as a basis for determination of areas to be ponded, it was decided to relate this factor to the number of bumps occurring in the unponded portions of the southbound main lanes. Figure 6 shows that the number of bumps can be expected to increase as PVR increases. The heavy dashed line in Figure 6 shows that when PVR exceeds 2 in. (5 cm) a rapid increase in the number of bumps per mile can be expected. Because this chart is based on a 7-year study, it does not clearly

Figure 5. Bumps recorded between 1958 and 1965.

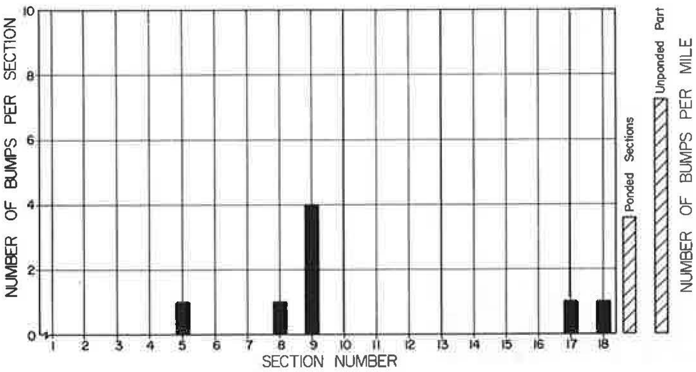


Figure 6. Relation of number of bumps for unponded portion of SBML after 7 years of traffic to average PVR.

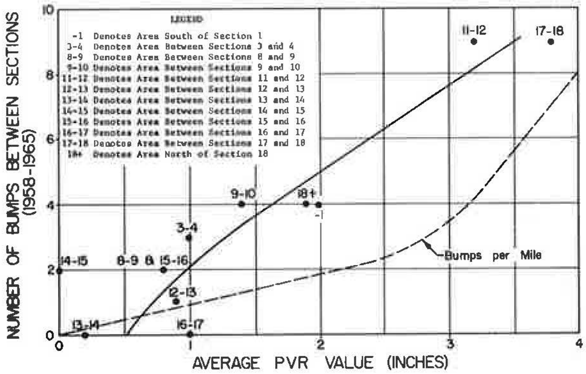


Figure 7. Relation of original PVR to movements as measured by levels.

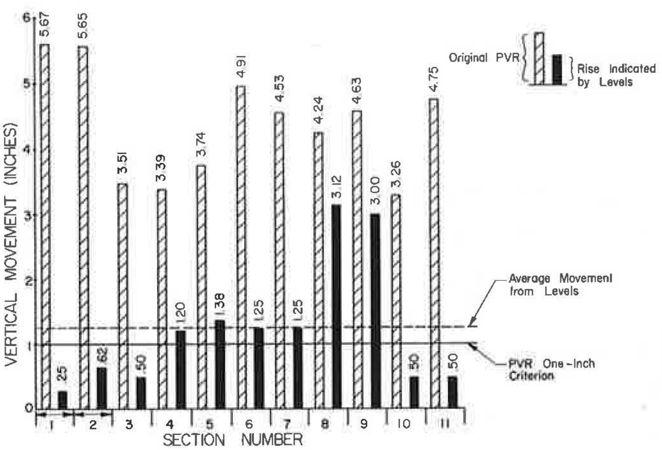


Figure 8. Average thickness of asphaltic concrete overlay (1971).

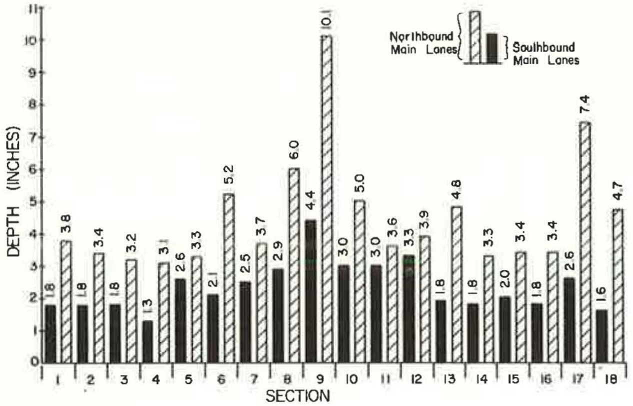


Figure 9. Maximum variation in thickness of asphaltic concrete overlay.

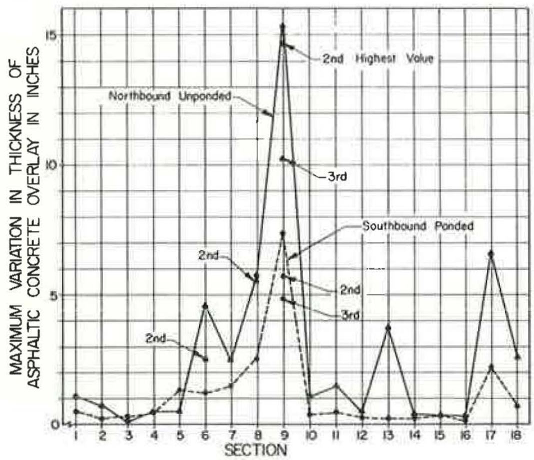
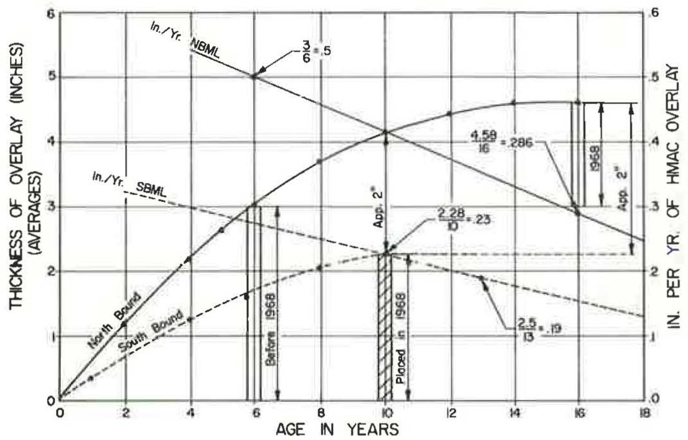


Figure 10. History of hot-mixed asphaltic concrete leveling.



indicate the magnitude of the minimum PVR for design, but the figure is probably somewhere between ½ and 1 in. (1.3 and 2.5 cm).

Figure 7 shows the original PVR values in relation to level measurements of movements recorded in May 1965. Paving grade level notes for ponded sections 1 through 11 were all that could be found in the files. This chart shows the original PVR values, which vary from 3.26 to 5.67 in. (8.28 to 14.4 cm). It may be noted that the average movement as measured from levels is approximately 1¼ in. (3.2 cm), whereas the target for ponding was 1 in. (2.5 cm).

In 1971 it was agreed that probing for thicknesses of AC overlay would be helpful in studying the manner in which the pavement performed. Figure 8 shows the average thickness of overlay for each section. Section 9 (the section with ditch drop inlets connected to underdrains) had the greatest average overlay thicknesses of any section. The overlay thicknesses are much greater for the NBMLs than they are for the SBMLs. This is as expected since the NBMLs are about 6 years older; however, a difference of 3 to 8 in. (7.6 to 20 cm) for several sections seems unusual, to say the least.

Figure 9 shows that the maximum variation in thickness (within sections) of AC overlay of the northbound main lanes is greater than that of the southbound main lanes in nearly all sections. This means that the original slabs of the SBMLs are not out of grade so much as those of the NBMLs. It should be kept in mind that the NBMLs are not the same age as the SBMLs. Points marked 2nd and 3rd represent the second and third highest values of thickness variation. They are presented to show that the peak values are not unusual values.

As stated before, it is difficult to compare overlay thicknesses of the NBMLs and SBMLs because of the difference in their service lives. In an attempt to place these data in their proper perspective, the histogram shown in Figure 10 was prepared. The coefficients for the depth of overlay per year were determined by dividing the thickness of overlay by the age of the road at the time the overlay was placed. Points for the curved lines were determined by multiplying the age in years by the corresponding coefficients taken from the scale on the right edge of the chart. Given that both the northbound and southbound lanes are of adequate structural strength to carry the traffic loads (there were no overload failures), it appears that the use of a combination of ponding and lime stabilization required approximately 2 in. (5 cm) less AC overlay during the first 10 years.

CONCLUDING REMARKS AND RECOMMENDATIONS

Moisture content tests taken before, during, and after ponding indicate the following.

1. Moisture from ponding did not penetrate the subgrade more than 4 ft (1.2 m) downward during a period of 24 days. A study of the horizontal movement of moisture was not made, but in one instance ponding was believed to have caused the tilting of the northbound main lanes. There was a distance of 20 ft (6.1 m) between the edge of the pond and the edge of the portland cement concrete.
2. Moisture contents at depths of 16 to 20 ft (4.9 to 6.1 m) began to increase after a period of several days' ponding and continued to increase all the way up to the 4-ft (1.2-m) level [6 ft (1.8 m) below finished grade] within a period of approximately 24 days. Although no data were taken to prove it, the vertical travel of moisture (up or down) was probably somewhat dependent on elevation of water tables.
3. Tests indicate that moisture contents below the pavement in ponded sections have remained fairly constant for 13 years since placement of the pavement. There has been some fluctuation of moisture contents at various depths, but it is believed that these have not been sufficient to cause severe movement of pavement in most of the ponded sections. Moisture content of samples taken in 1972 below the cement-stabilized shoulders, which had severely cracked away from the edge of the portland cement concrete, was very erratic. Such samples are probably not representative of conditions below the portland cement concrete.
4. The moisture contents found under the pavement at various depths, before the

shoulders cracked away from the concrete, are in fairly close agreement with the desired moisture contents calculated in accordance with the method given in the complete report (3).

The maximum movements measured by profile levels are in general agreement with the movements predicted by use of the potential vertical rise method for after ponding conditions. The PVR method was very useful in helping to select locations for ponding and determining moisture contents required before termination of ponding.

The bump surveys made after the SBML pavement was 7 years old showed that the ponded sections had only one-half as many bumps per mile as the unponded portion of the same lanes. Of a total of eight bumps occurring in all ponded sections, one-half of these occurred in section 9. Strangely enough this is the only section on the project where underdrains were connected to drainage ditch drop inlets that were supposed to handle fairly large drainage areas. If the bumps in this section were excluded from the data, there would have been four times as many bumps per mile in the unponded portion of the project as in the ponded sections.

A study of the overlay leveling applications shows the following:

1. The leveling overlay thickness for the unponded NBML sections is considerably thicker than it is for the ponded sections in the southbound lanes.
2. The roughness of portland cement concrete slabs, as measured by maximum variation in overlay thickness, shows that 14 of the 18 sections contain rougher slabs in the unponded lanes than in the southbound ponded lanes.
3. The validity of conclusions 1 and 2 is in jeopardy because the northbound pavement lanes are 6 years older than the southbound lanes. Accordingly, a histogram of overlay thicknesses was made, and it showed that ponding and lime stabilization of swelling subgrades can be expected to reduce the required depth of AC overlay by approximately 2 in. (5 cm) within the first 10 years of pavement life.

In general, it is concluded that ponding and lime stabilization of subgrade were highly successful in preventing heaving in all sections except the one section where underdrains were connected to ditch surface drainage by use of drop inlets.

A study of the unponded areas between and beyond the ponded sections (Table 2 and Figure 6) indicates that, if ponding had been used more extensively and if underdrains had not been connected to ditch drop inlets, probably only five of the 43 bumps recorded would have occurred in these areas. Table 2 indicates that the higher the PVR is the greater is the number of bumps expected and that if few to no bumps are desired a PVR criterion of ½ in. (1.2 cm) should be used. If this criterion had been followed during construction, only two bumps should have occurred in the ponded sections and five in the remaining portions. If only seven bumps had occurred on the entire length of the SBML, overlaying the entire project with hot-mixed AC probably would not have been necessary for many more than 10 years.

The foregoing conclusions appear to justify the following recommendations.

1. For all subgrades with PI in excess of 35, calculate potential vertical rise values and determine whether ponding of subgrade is necessary and feasible. PVR data should be used to calculate desired moisture contents at various depths below pavement to determine when to cease ponding.
2. The use of ponding should be given serious consideration where a heavy traffic facility is involved and it has been determined that ponding is necessary and feasible.
3. If ponding is used, every effort should be made to prevent evaporative drying before placement of the pavement. One of the most practical ways to accomplish this is by the use of a wide belt of lime-stabilized subgrade. The use of this stabilizer allows the work to proceed before excessive drying takes place. In addition, the lime-treated subgrade helps form a strong working table and if extended widely enough makes an excellent barrier to evaporative drying and shrinking. Various granular materials will also decrease evaporation but usually will not form a strong working table unless placed in very thick layers.

4. Underdrains should be used sparingly in swelling soils, and they should not be connected to drainage ditch drop inlets.

5. For all future projects in swelling soils, ponding should be used more extensively than was done on this project, and its use in conjunction with deep plow mixing of lime should be investigated.

6. Special effort should be made to prevent cracks formed by shoulders shrinking away from the edges of pavement.

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So much information and data have been collected from so many people over such a long period of time that it is difficult to name those who have contributed to this project, but the authors are grateful to all who have given assistance and encouragement to the performance of the experiments and the preparation of this report.

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A STUDY OF SOIL CEMENT WITH CHEMICAL ADDITIVES

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The best practical method of stabilization for construction of flexible pavements in Rhode Island was determined by investigating 17 soils with nine different chemical additives. The study was conducted both in the laboratory and in the field. The laboratory study dealt mainly with the selection of the most effective chemical additive for the cement-stabilized Rhode Island soils. The field study was essentially an evaluation of the effectiveness of cement stabilization with and without a chemical additive based on pavement performance. Results indicated that sodium sulfate is the most effective chemical additive. Addition of 1 percent sodium sulfate can significantly increase strength and durability and decrease frost heaving. Pavements containing soil cement plus 1 percent sodium sulfate base possess greater rigidity than those with soil cement alone. Soil cement test pavements developed cracks; an addition of 1 percent sodium sulfate did not appear to significantly influence the cracking behavior of pavement.

•BECAUSE of its successful engineering performance and low cost, soil cement has been used increasingly as a base course material in highway pavements. However, soils are not equally effective for cement stabilization; some soils react poorly with cement and are economically unfeasible for pavement construction. During the last decades, research has been focused on improving soil cement properties by the use of chemical additives (1-7). The main objective of these studies was to enhance the effectiveness of portland cement so that either the quantity of cement required can be reduced or the soils that cannot be stabilized economically with cement alone can be stabilized with the addition of a trace amount of chemical additive.

Among the studies on chemical additives, Catton and Felt (1) evaluated the effectiveness of calcium chloride based on the compressive strength and wet-dry and freeze-thaw test data and concluded that strength can be increased significantly with a small quantity of calcium chloride. Lambe and coworkers (2-5) investigated the compressive strength of cement-stabilized soils with several kinds of chemical additives including sodium compounds. They found that a trace amount of chemical additive could be either beneficial or detrimental to the soil cement depending on the types of soil and chemical additive. Among their findings, sodium sulfate was uniquely effective on sandy soils with organic matter. With silty soils, the strength increase due to sodium additives was smaller at higher cement contents. The effectiveness of sodium compounds on soil cement varied with soil type and decreased with an increase in soil plasticity or organic matter content of the soil or both. For a silty soil, the effectiveness of sodium compounds decreased in the order sulfate, aluminate, metasilicate, carbonate, and hydroxide. In their discussion of the paper by Lambe et al. (3), Norling and Packard (6) reported freeze-thaw and wet-dry durability test results and concluded that the effects of the additive on durability were similar to the effects on compressive strength.

Laguros and Davidson (8) evaluated the effect of compounds of sodium, calcium, magnesium, and commercial lime on the strength property of soil cement for eight

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soils from different horizons. Their results indicate that organic topsoils benefited from the incorporation of sulfates when the soils were acidic and low in clay content. With increasing clay content and an alkaline environment, the addition of calcium and magnesium ions generally resulted in greater strength. In addition, the results of their durability tests verified the strength benefaction derived from adding the chemicals to soil cement mixtures.

The preceding information shows that whether the soil cement can be significantly enhanced depends essentially on the types of soil and chemical additive. The many factors influencing the behavior of admixtures make it virtually impossible to determine the effectiveness of chemical additives for all soils without independently investigating each soil. In addition, nearly all available information was derived from laboratory studies. The complex environment in the field suggests that a successful evaluation of the property of soil cement mixture relies greatly on the field test. All of these constitute the need of the study reported here.

The primary objective of this study was to determine the feasibility of using cement-stabilized local silty soils and also to determine the best practical method of stabilization for road construction in Rhode Island. The study was conducted in two phases. The first phase mainly was a laboratory investigation that emphasized the selection of the most effective chemical additive and determination of treatment level. The second phase was a field study mainly to evaluate the effectiveness of cement stabilization with and without a chemical additive based on test pavement performance.

MATERIALS

The soils studied are typical of glacial till and outwash deposits available in various locations throughout Rhode Island. The area is unique in that, although the whole of it had been subjected to glaciation, almost all clay sizes had been washed out to the sea. Based on the soil map developed by Moultrop (9), 17 soil samples were selected for laboratory investigation. The physical properties of the test soils are given in Table 1. There are essentially only two classes of soils: A-2-4 and A-4. Type I portland cement was used throughout. Distilled water was used for mixing in the laboratory, and tap water was used in the field study. The trace chemicals studied are as follows:

Name	Formula
Sodium sulfate	Na_2SO_4
Sodium aluminate	$NaAlO_2$
Sodium carbonate	Na_2CO_3
Sodium metasilicate	$NaSiO_3 \cdot 9H_2O$
Sodium phosphate	Na_2PO_4
Lithium fluoride	LiF
Sodium fluoride	NaF
Quadrafos	$(NaPO_3)_4$
DAXAD	—

The selection of trace chemicals resulted primarily from the research findings of earlier studies by Lambe et al. (1, 2, 3, 10).

LABORATORY TEST PROCEDURES

All test soils were air dried, pulverized, and screened through a No. 10 sieve. Mixing was done in a Hobart kitchen mixer at a low speed. First the dry soil and cement were mixed for 30 sec, and then the molding water and trace chemicals were mixed for 1 min.

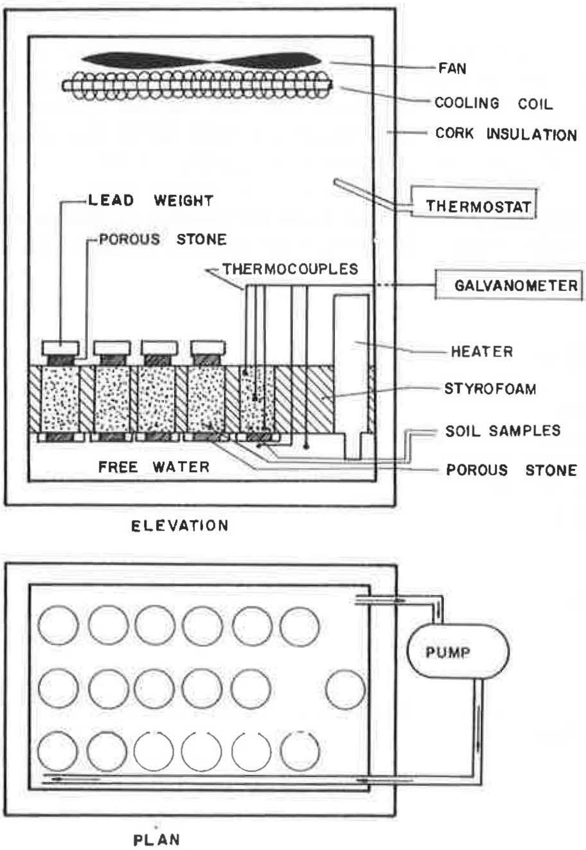
Table 1. Properties of test soils.

Soil	Sand[a] (percent)	Silt[a] (percent)	Clay (percent)	Liquid Limit	Plastic Limit	Maximum Dry Density[b] (lb/ft^3)	Optimum Moisture Content (percent)	Medium Grain Size (mm)	Uniformity Coefficient	AASHO Classification
1	79	21	0	NP	NP	116.0	13.3	0.240	7.4	A-2-4
2	72	28	0	NP	NP	102.3	17.9	0.120	4.5	A-2-4
3	78	21	1	28.2	22.8	117.6	15.6	0.30	24.5	A-2-4
4	80	20	0	NP	NP	117.0	13.8	0.28	12.6	A-2-4
5	78	21	1	NP	NP	118.1	12.0	0.155	11.6	A-2-4
6	72	27	1	NP	NP	118.0	16.0	0.190	11.9	A-2-4
7	45	48	7	30.2	20.1	106.8	16.5	0.068	7.1	A-4
8	40	53	7	29.0	19.5	106.0	17.2	0.055	4.6	A-4
9	68	32	0	NP	NP	107.1	17.5	0.068	13.5	A-2-4
10	12	86	2	27.0	20.2	100.5	21.0	0.020	3.2	A-4
11	30	70	0	26.5	21.3	98.5	21.0	0.040	3.5	A-4
12	31	69	0	26.9	20.8	103.5	19.1	0.048	2.9	A-4
13	50	50	0	NP	NP	105.5	17.6	0.078	4.7	A-4
14	39	61	0	25.8	19.9	101.6	19.4	0.034	3.5	A-4
15	17	81	2	27.1	20.3	102.4	19.7	—	3.9	A-4
16	56	44	0	NP	NP	109.0	15.0	0.12	13.5	A-4
17	1	99	0	28.5	20.5	101.0	19.1	0.018	2.4	A-4

Note: 1 lb/ft^3 = 16 kg/m^3.

[a]ASTM-ASCE grain size scale. [b]Standard Proctor with cement plus 1 percent Na_2SO_4.

Figure 1. Frost heave test apparatus.



Specimens were molded in a Harvard miniature mold, and density and water content corresponded approximately to the optimum moisture content and maximum dry density of a standard Proctor test, AASHO T-99-49. Sufficient samples were made to provide at least three values for the immersed unconfined compression test and the durability and laboratory frost heave tests. All specimens were cured at approximately 100 percent relative humidity and 70 F (21 C) for various periods of time.

The soil cement specimens were tested for compressive strength after being moist cured for 7 and 28 days. Specimens were soaked 24 hours and then loaded at a constant rate of strain of 0.05 in./min (1.3 mm/min) to failure.

The durability of soil cement specimens was evaluated by using the standard wet-dry test, AASHO T-135, and the freeze-thaw test, AASHO T-136. The weight loss of the test specimens was determined after 12 cycles.

The frost heave test was conducted in a cold chest (Figure 1). The test specimens, 1.4 in. (35 mm) in diameter by 3.0 in. (76 mm) high, were first cured for 28 days and then placed in the frost chest where they were subjected to 12 cycles of freezing for 2 days at 27 F (-2.8 C) and 1 day of thawing. Freezing conditions were applied only to the top face of the soil, and the bottom face was in contact with water at a temperature of 42 F (5.6 C). The apparatus could accommodate 17 specimens at a time. Specimens were frozen at approximately two-thirds of their height.

Each test soil was stabilized with a number of cement contents. The minimum cement content was determined based on compressive strength requirements, wet-dry and freeze-thaw weight loss limits, and tolerable frost heave values. The minimum compressive strength requirement was based on the Portland Cement Association's 7-day wet unconfined compressive strength criterion, which was expressed in terms of percentage of silt size content. Maximum weight losses of 14 percent for A-2-4 soils and 10 percent for A-4 soils as established by the Portland Cement Association were used as criteria for durability. The criterion for frost heave was quite tentative. Certainly more heave meant more imbibed water, hence less thawed strength. On the basis of a water content change of approximately 4 percent, 3 percent heave was regarded as the indicator of inadequate resistance to frost action.

LABORATORY TEST RESULTS

About one-half of the chemicals studied resulted in an increase in unconfined compressive strength above that obtained by treating the soil with portland cement alone. Figure 2 shows the effect of additive on the 7-day strength for soils 6 and 17. Overall, the chemical that caused the best consistent response for all test soils was sodium sulfate, and the amount of sodium sulfate required to produce maximum efficiency was 1 percent by dry weight of soil.

The 28-day unconfined compressive strength of soil cement with and without 1 percent sodium sulfate for 17 test soils is given in Table 2. Of the 17 soils tested, all but three (soils 2, 10, and 17) benefited significantly from the additive. Although the additive has no significant effect for soil 10 at 6 and 10 percent cement content and essentially no effect for soil 17 at 6 percent cement, it is detrimental to the development of 28-day strength of soil 2 at both cement contents and of soil 17 at 10 percent cement. Of the soils that benefited from the additive, only soils 8, 9, and 12 show a greater percentage of strength increase at higher cement content. For the rest of the soils, the strength increase due to sodium sulfate is small at higher cement content, confirming the finding of Lambe et al. (3).

Table 2 also gives a wide range of variation—from -31 to about 770 percent strength increase—in the effectiveness of sodium sulfate on the 28-day unconfined compressive strength of the test soils. An attempt was made to correlate the plasticity index with the percentage of strength increase; unfortunately, no clear relation was obtained. The organic content of the test soils ranges between 0.3 and 0.7 percent by weight, and the pH value varies approximately from 5.0 to 5.8. No conclusion regarding the effect of organic content and pH on the strength increase was found. Other physicochemical factors such as cation exchange capacity and clay mineral composition may have a

Figure 2. Effect of chemical additives on 7-day wet compressive strength of soil cement.

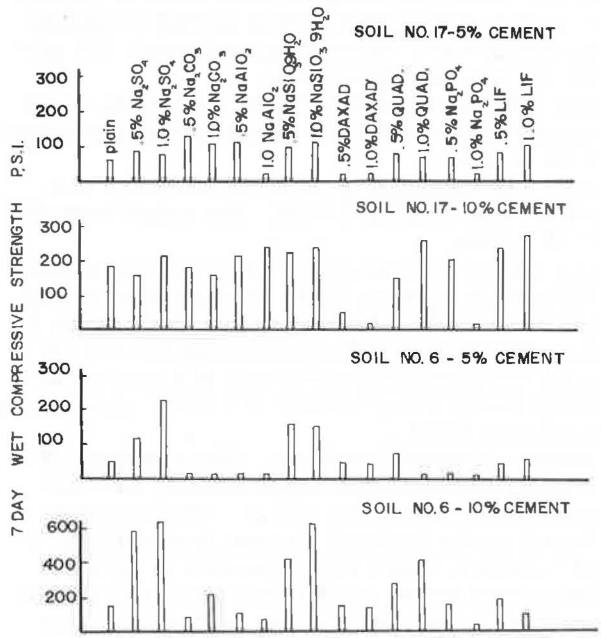


Table 2. Effect of sodium sulfate on 28-day unconfined compressive strength.

Soil	Cement Content (percent)	Unconfined Compressive Strength (psi): Cement Only	Unconfined Compressive Strength (psi): Cement + Na_2SO_4	Percentage of Strength Increase[a]	Soil	Cement Content (percent)	Unconfined Compressive Strength (psi): Cement Only	Unconfined Compressive Strength (psi): Cement + Na_2SO_4	Percentage of Strength Increase[a]
1	6	50	235	370	10	6	100	90	-10
	10	402	510	27		10	235	240	2
2	6	202	173	-14	11	6	28	136	386
	10	385	266	-31		10	96	238	148
3	6	56	205	266	12	6	50	200	300
	10	124	361	191		10	75	440	486
4	6	60	275	358	13	6	50	118	136
	10	400	418	5		10	115	255	122
5	6	72	402	458	14	6	74	112	51
	10	147	600	308		10	130	182	40
6	6	112	524	368	15	6	20	36	80
	10	195	766	293		10	50	72	44
7	6	33	275	732	16	6	54	183	239
	10	57	440	672		10	137	264	93
8	6	50	215	330	17	6	143	143	0
	10	100	510	410		10	235	175	-26
9	6	40	250	525					
	10	75	650	766					

Note: 1 psi = 6.9 kPa.

[a]Percentage of strength increase = $\frac{\text{(strength of cement plus 1 percent } Na_2SO_4\text{) - (strength of cement alone)}}{\text{strength of cement alone}} \times 100.$

decisive influence on the effectiveness of sodium sulfate. No data are, however, available to make further conclusions possible.

The test results indicate that soils of the same classification do not respond equally to a chemical additive such as sodium sulfate. Thus, the effectiveness of a trace additive cannot be evaluated based on soil classification.

Only 10 of 17 test soils were studied under standard PCA freeze-thaw and wet-dry tests. Soils for durability tests were selected to cover a wide range of gradation and plasticity characteristics of each class of soil. In the durability study, two cement contents, 7.5 and 10 percent, were used. A 7.5 percent cement content rather than 6 percent as used in the wet strength tests was adopted because, at this treatment level, the weight loss under durability tests can be kept as close as 14 percent for A-2-4 soils and 10 percent for A-4 soils, which are the durability criteria established by the Portland Cement Association. Results of the durability study given in Table 3 indicate that addition of 1 percent sodium sulfate to the soil cement resulted in a decrease in the percentage of weight loss for most soils studied. Some soils, especially soils 1 and 8, however, do not benefit from the trace additive.

A comparison of Table 3 with Table 2 reveals that although addition of 1 percent sodium sulfate increased the strength of soils 1 and 8, their durability was not necessarily increased by addition of the same chemical. In other words, a trace additive does not necessarily always improve both strength and durability of a soil cement simultaneously. This result emphasized that the effectiveness of a trace chemical must be determined based on not only its strength property but also the durability of the mixture.

Another measure of the resistance of soil cement to frost action was studied by measuring frost heave. The test apparatus is shown in Figure 1. Because considerable time was lost in designing and constructing the apparatus and each test series is very time-consuming, 40 to 45 days, only five of 17 test soils were studied. Figure 3 shows a typical frost heave for soil 9; the figure indicates clearly the effectiveness of 1 percent sodium sulfate in reducing frost heaving. The results of the frost heave study are given in Table 4 in terms of the minimum cement requirement based on a criterion of 3 percent heaving; the table also gives the minimum cement requirement based on strength, freeze-thaw, and wet-dry durability tests. These minimum cement requirements were used as a basis for designing experimental pavements for the field study.

TEST ROAD

The test road is a two-lane highway with 10-ft (3-m) shoulders and is essentially a section of RI-214, a state secondary highway in Middletown, Rhode Island. It is composed of two control sections of conventional design and five experimental sections of different materials in base and subbase courses as shown in Figure 4. All sections are surfaced with 3 in. (76 mm) of bituminous concrete.

Specifications for construction of the test pavements were modified from PCA suggestions; detailed specifications were given in the initial preconstruction report.

The base and subbase course materials were pulverized and mixed in place with a Trav-L-Plant multiple-pass rotary mixer. A bulk cement truck with a compressed-air distributing system and a pressure distributor truck were used for applying cement and water to the roadway. Normally, one section of subbase or base was constructed per day. Each section was processed in 5 or 6 strips, 8 to 9 ft (2.4 to 2.7 m) wide; compaction of one strip was concurrent with processing of the adjacent one. Sodium sulfate was dumped in to make a 20 percent solution. The solution was kept circulating in the tank.

A light steel-wheeled roller was used on the initial compaction pass to push down stones; subsequent compaction was with a pneumatic roller. In all cases, compaction was above the specified 95 percent, and an average of 99 percent maximum density was attained.

Table 3. Results of freeze-thaw and wet-dry tests.

Soil	Cement Content (percent)	Percentage Loss With 1 Percent Na_2SO_4		Percentage Loss Without Na_2SO_4	
		Wet-Dry	Freeze-Thaw	Wet-Dry	Freeze-Thaw
1	7.5	5.25	4.29	2.91	2.51
	10.0	3.86	4.50	3.44	2.00
5	7.5	5.22	7.02	7.77	8.65
	10.0	1.21	1.62	4.77	3.32
6	7.5	7.15	8.35	4.65	12.12
	10.0	1.95	2.46	3.31	2.05
8	7.5	10.52	9.65	2.85	5.13
	10.0	3.63	6.30	2.62	1.92
10	7.5	12.05	13.62	11.62	100.0
	10.0	9.78	11.75	9.77	10.75
11	7.5	1.65	100.0	2.62	5.31
	10.0	1.65	1.67	3.13	5.11
12	7.5	7.49	14.30	6.62	26.1
	10.0	7.65	3.70	3.38	1.22
13	7.5	3.39	6.00	6.00	100.0
	10.0	2.99	7.31	5.32	100.0
16	7.5	1.61	5.60	6.30	100.0
	10.0	1.02	0.0	2.75	100.0
17	7.5	10.75	15.12	18.55	18.62
	10.0	8.65	10.05	7.23	10.17

Figure 3. Effect of sodium sulfate on frost heave for soil 9.

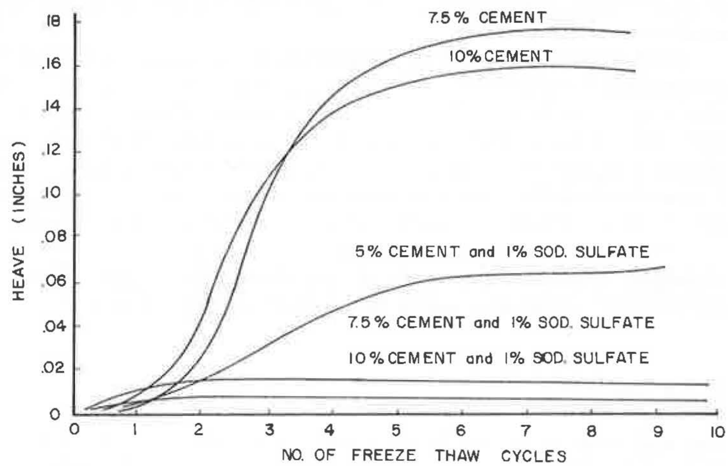


Table 4. Minimum cement requirements (in percentage of dry soil weight) with 1 percent sodium sulfate.

Soil	Based on 7-Day Strength	Based on Freeze-Thaw	Based on Wet-Dry	Based on 3 Percent Frost Heave
1	7.0	5.5	5.5	—
2	11.5	—	—	—
3	9.0	—	—	—
4	7.4	—	—	—
5	3.5	5.0	5.0	—
6	3.5	5.5	5.0	5.0
7	6.0	—	—	—
8	6.3	7.5	7.3	7.0
9	6.0	—	—	5.8
10	10.0	13.5	12.0	—
11	11.2	7.5	6.0	—
12	7.5	7.0	8.5	—
13	10.0	6.0	5.5	—
14	13.0	—	—	7.5
15	8.4	—	—	7.5
16	11.0	5.5	5.5	—
17	10.4	13.5	12.0	—

TEST ROAD PERFORMANCE

The performance of test pavements was evaluated by using the Benkelman beam test, plate bearing test, and roughometer test and on the basis of cracking and response to freezing temperature. Additional information regarding pavement performance evaluated before 1972 is reported elsewhere (11). The following discussion deals mainly with the relative performance of the sections containing soil cement with and without sodium additive.

The Benkelman beam tests were conducted annually in the spring and summer at the locations shown in Figure 5. The test truck had a rear axle load of 18 kips (8154 Kg) and a tire pressure of 80 psi (550 kPa). Detailed testing procedures are given by Roderick and Huston (12).

Test results are shown in terms of mean maximum deflection in Figure 6; each deflection value is the average of 16 measurements. Comparison of the deflection in sections 5, 6, and 7, which contain the same amount of cement in the subbase layer, shows that section 6 (with 6 percent cement plus 1 percent sodium sulfate base) deflects as little as section 5 (with 11 percent cement base), indicating the beneficial effect of the addition of 1 percent sodium sulfate. However, section 7 (with 10 percent cement plus 1 percent sodium sulfate) unexpectedly deflects as much as section 6. A possible reason for this is that the subbase material in section 7 has only about half the strength of the material in section 6 (Figure 7). Figure 7 shows the unconfined compressive strength of the base and subbase material used in the experiment sections. Note that there are two types of specimens tested for base material: One is a 1.4-in.-diameter (35-mm) by 2.8-in.-high (71-mm) laboratory specimen compacted to the same moisture content and density as those in the field and cured in a moist room, and the other is a 4-in.-diameter (102-mm) by 8-in.-high (204-mm) undisturbed core. The greater strength of the base course material in section 7 as compared with that of section 5 verifies further the beneficial effect of sodium sulfate.

The plate bearing tests were conducted annually in different seasons by using a 12-in.-diameter (305-mm) plate at two permanent test sites in each section. The test sites are located on the boundary separating the traffic and shoulder lanes, 150 ft (46 m) from the beginning of each section on the right and 150 ft from the end on the left. A hydraulic jack was used to apply static plate load, and two 0.001-in. (0.025-mm) dial gauges at opposite ends of a diameter were used to measure plate deflection. Test results shown in Figure 8 generally follow those of the Benkelman beam test.

The roughometer tests were conducted by the Rhode Island Department of Transportation by using the BPR roughometer. No conclusive difference in the pavement roughness between experimental sections due to addition of sodium sulfate has been found.

Cracking behavior of the experimental sections was carefully surveyed and mapped annually. The depths of the cracks were determined by taking core samples across a crack. It was found that the cracks generally go through the entire base course but only to about middepth of the subbase course. Except for some transverse cracks that developed along the construction joints between sections and that had widths of about $\frac{1}{4}$ to $\frac{3}{8}$ in. (6.3 to 9.5 mm), the widths of the cracks were in general smaller than $\frac{1}{8}$ in. (3.2 mm), and once cracks developed no further change in width was noted.

The surface crack pattern mapped in March 1972 is shown in Figure 9. It is interesting to note that all surface cracks developed in transverse and longitudinal directions only. Some transverse cracks apparently developed along the construction joints between sections, and longitudinal cracks appeared to develop along the joints of the strips produced during construction. Most cracks are, however, primarily caused by cement hydration and thermal stress.

Figure 10 shows the crack length surveyed. Cracking increases with time for all experimental sections. The rate of increase in cracking, however, varies for various sections. Data obtained in April 1974 indicate that (a) section 3 (9 percent cement) developed the greatest amount of longitudinal cracks (most of the cracking, however, appeared along the centerline of the pavement) and section 6 (6 percent cement plus 1 percent sodium sulfate) developed the least; (b) transverse cracks developed the most

Figure 4. Bases and subbases of experimental road sections.

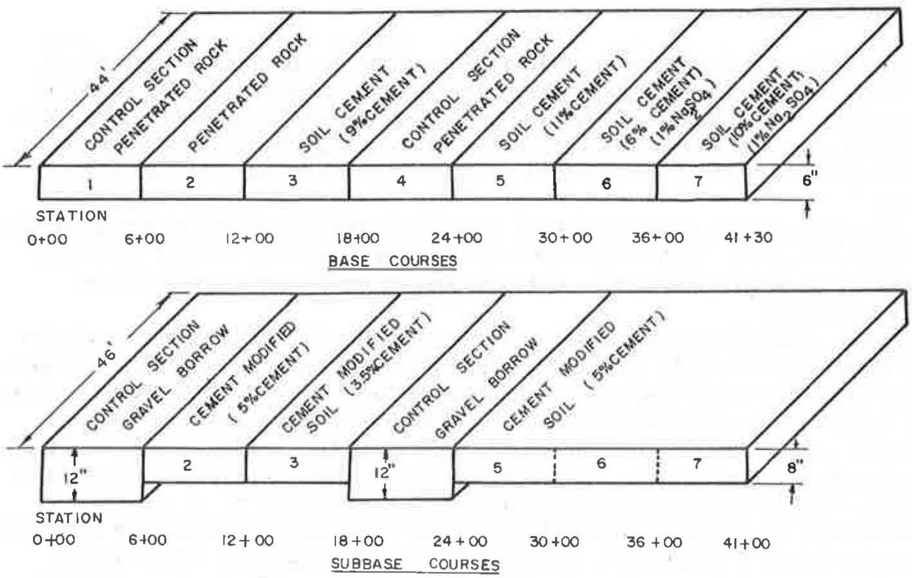


Figure 5. Layout of Benkelman beam deflection test sites.

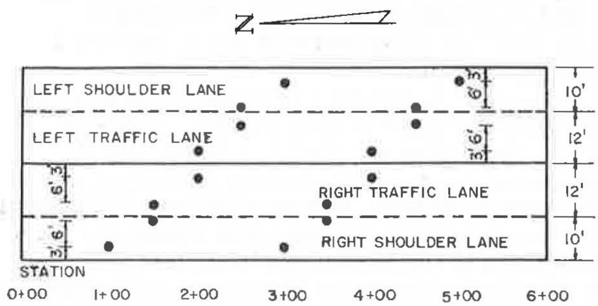


Figure 6. Mean maximum pavement deflection in Benkelman beam test.

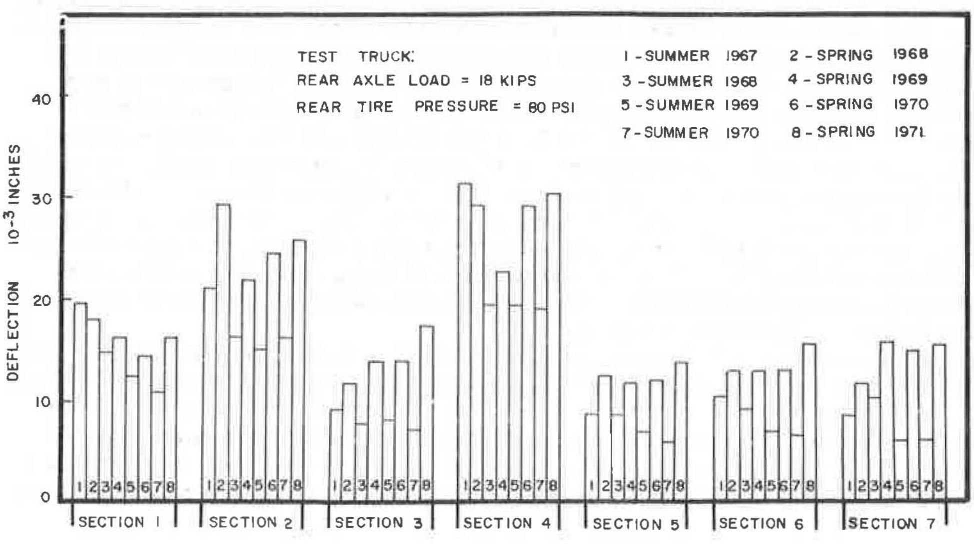


Figure 7. Unconfined compressive strength of base and subbase materials.

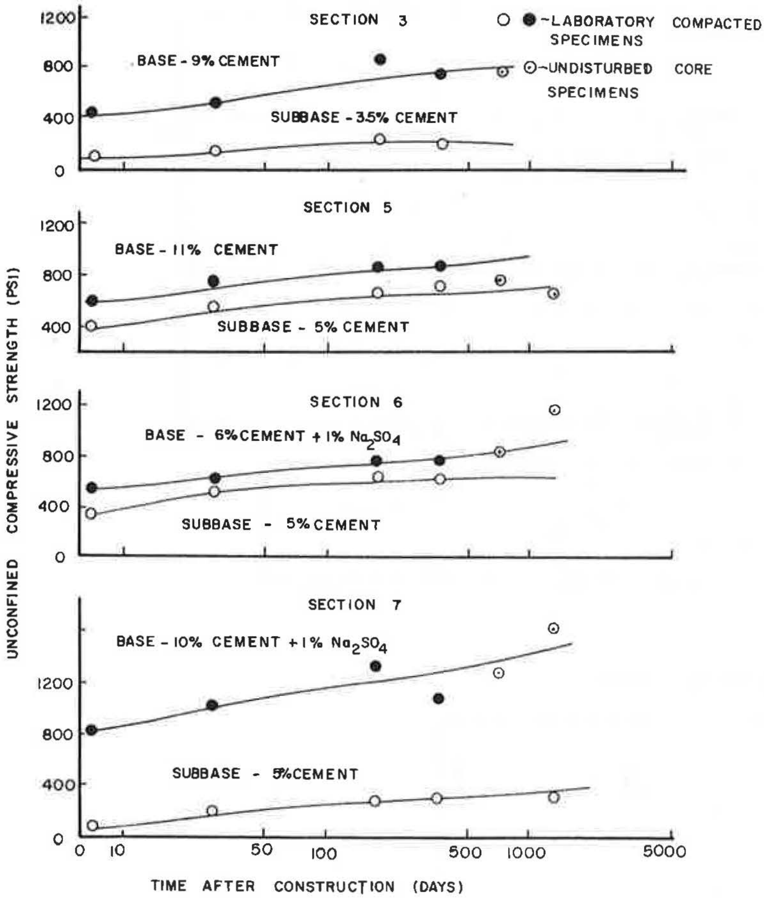


Figure 8. Pavement deflection under 15-kip (67-kN) plate load.

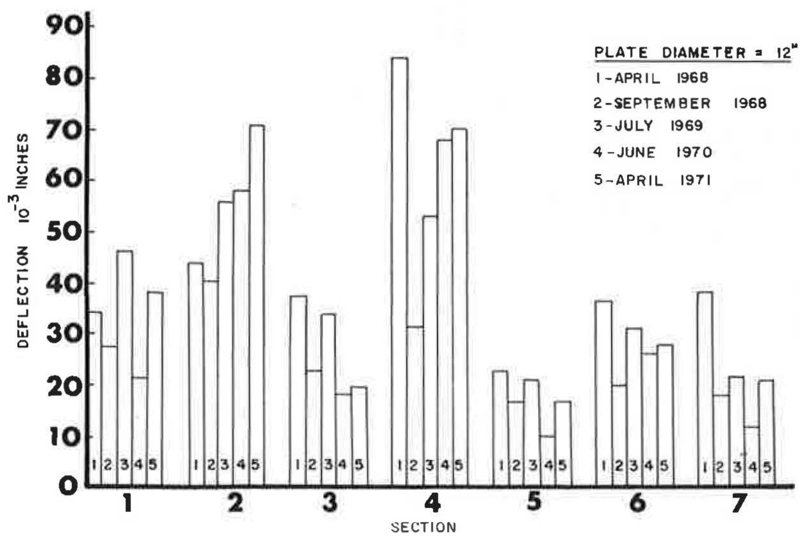


Figure 9. Cracks pattern mapped on March 28, 1972.

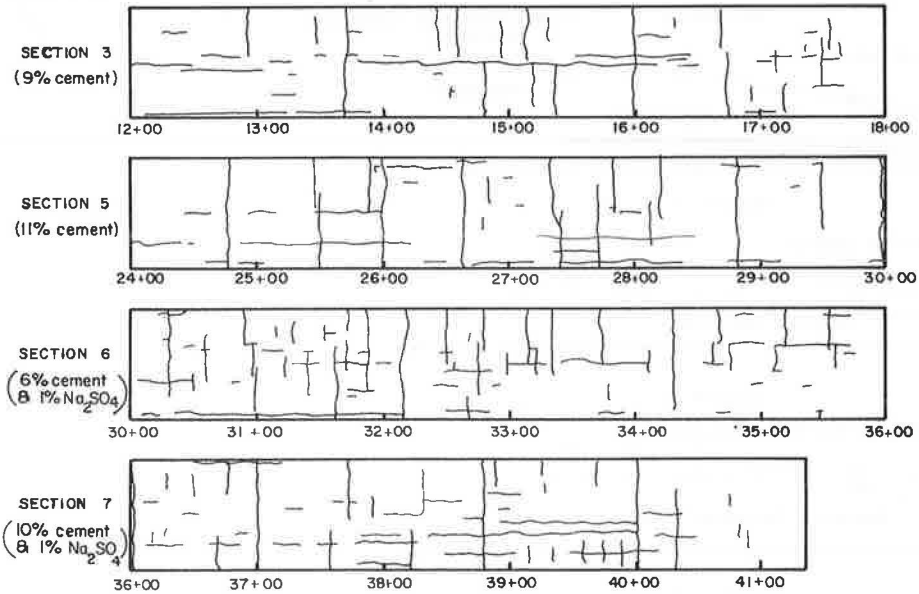
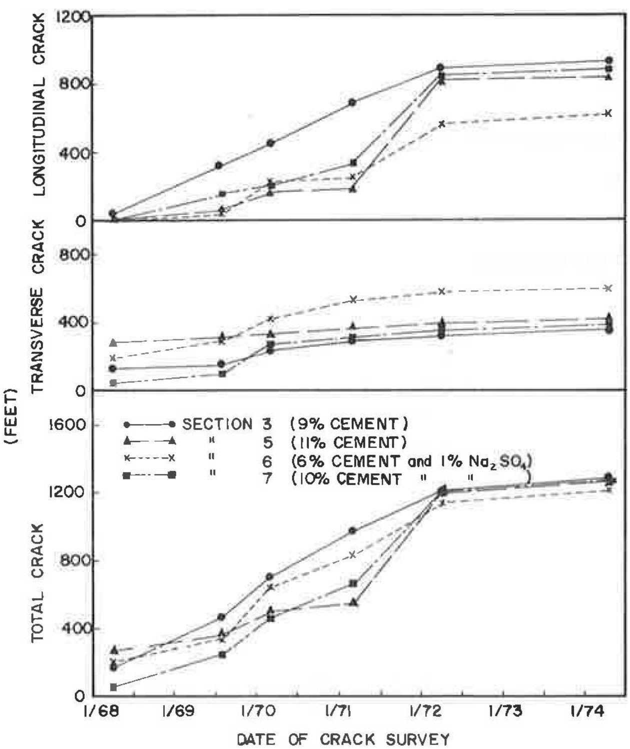


Figure 10. Length of surface crack in test sections.



in section 6 and the least in section 3; and (c) the total length of the cracks was shortest in section 6 and was almost equal in the other three sections. The results obtained to date seem to suggest that cracking of the experimental pavement tends to level off after 6 years' service; the intensity of cracking is greater at higher cement content. The addition of 1 percent sodium sulfate does not influence the cracking behavior significantly, although the strengths are increased considerably as has been reported.

The response of the experimental pavements to freezing temperature has been reported elsewhere (11). No conclusion regarding the relative response of each soil cement section to frost penetration can be drawn from the data available.

SUMMARY AND CONCLUSIONS

The effect of chemical additives on the cement-stabilized Rhode Island soils was studied with 17 natural soils that are typical of glacial till and outwash deposits. The study was conducted both in the laboratory and in the field. In the laboratory, nine chemical additives of reagent grade were investigated by using strength, durability, and frost heave tests. Test results indicate that, of the nine chemicals studied, sodium sulfate generally improved the strength and durability of cement-stabilized Rhode Island soils better than the others. Furthermore, soils of the same classification do not respond equally well to sodium sulfate.

In the field, a test road was constructed to study the effectiveness of sodium sulfate in soil cement. The test pavements were evaluated according to their relative performance under loading, cracking behavior, and resistance to frost action. Under the same subbase support, the pavement containing soil cement plus a sodium sulfate base layer deflects as little as that containing a base layer without sodium sulfate but with greater cement content. The increase in pavement stiffness due to the addition of 1 percent sodium sulfate provides a further indication of the beneficial effect of sodium additive strengthwise. Results of the crack study seemed to suggest that pavements containing base course with a higher cement content developed more cracks; the addition of 1 percent sodium sulfate did not appear to influence cracking behavior significantly. The effect of sodium sulfate on the frost resistance of soil cement pavement could hardly be seen from the available field test data.

Based on the test results, it is concluded that sodium sulfate is the most effective chemical additive of those studied for the cement-stabilized Rhode Island soils. Addition of 1 percent sodium sulfate does not result in a significant influence on cracking behavior; however, it significantly improves strength, durability, and frost resistance of the soil cement.

ACKNOWLEDGMENTS

The authors wish to express their sincere thanks to the Rhode Island Department of Transportation and the Federal Highway Administration for their support of this study.

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USING ADDITIVES TO IMPROVE COLD WEATHER COMPACTION

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A laboratory investigation was conducted to determine possible methods of improving cold weather earthwork techniques and extending the construction season. Compaction tests, using standard and modified AASHO compactive effort, were conducted on a silty sand at temperatures of 68 and 19 F (20 and -7 C). The low-temperature tests of soil with 0, 2, and 3 percent calcium chloride were performed to study the influence of an additive on the moisture-density relationship of the test soil. Tests at 68 F (20 C) were used to establish a frame of reference for the low-temperature tests and to determine the normal compaction characteristics of the silty sand. To eliminate the effects of particle size, a single sized particle was prepared in the laboratory and used throughout the low temperature testing. Test results indicate that (a) additives can be effectively used to offset the detrimental effect of low temperature on compaction; (b) soils compacted at 19 F (-7 C) and treated with 2 or 3 percent calcium chloride have essentially the same compaction characteristics as an untreated soil compacted at a temperature of 68 F (20 C); and (c) an untreated soil compacted at 19 F (-7 C) has significantly lower dry densities than a soil compacted at 68 F (20 C). The low densities that occur when a soil is compacted while frozen are due to the formation of ice within the pore spaces. Modeling the pore fluid as a solution of additive and water and using the concept of phase equilibria shows that the amount of additive required to prevent the formation of ice in a compacted soil is related to the freezing point depression characteristics of the additive. A discussion of field problems and application techniques is included to aid the practicing engineer in using the method suggested in the paper.

•MANY innovative techniques and practices have been developed in the area of winter construction, particularly in the placement of concrete masonry. However, most state and federal agencies still prohibit the placing of frozen soils in embankments or fills. Yoakem (15) researched the public and private policy associated with excavation, placement, and compaction of soils for embankments and foundations and reported that "Twenty-five of the forty-five highway departments which replied to the questionaire stated they do not construct embankments using frozen soils during freezing weather and they do not allow footings or pavements to be placed on frozen ground."

The limitations on cold weather earthwork are due primarily to the observed difficulties of obtaining specified densities and the implied problems of large settlement and inadequate strength. For example, Higher, Altschaeffl, and Lovell (3) recognized that low-temperature compaction is approximately equivalent to reducing the effective compactive effort and found that a decrease in compaction temperature results in a decrease in unit weight, degree of saturation, and undrained strength. They noted that cold but unfrozen soil may be successfully field compacted by increasing the level of

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compactive effort. Other authors (1, 8) also noted higher densities when test temperatures were increased from near freezing to 75 F (24 C). Typically, increases in maximum dry density were from 3 to 5 lb/ft^3 (48 to 80 kg/m^3) for granular soils and as high as 11 lb/ft^3 (175 kg/m^3) for some fine-grained soils. Johnson and Sallberg (6) found that when a sandy soil was compacted a decrease in dry density of 2 to 3 lb/ft^3 (32 to 48 kg/m^3) occurred as a result of lowering the temperature from 75 to 40 F (24 to 4 C). They also cited results that clearly show the large reduction in soil unit weight caused by temperatures below 32 F (0 C).

The results of these investigations catalog the effect of temperature on soil compaction, but no attempt is made to investigate possible methods of offsetting the detrimental influence of low temperature. If soil could be successfully compacted in spite of below-freezing temperatures, it is possible that the construction season could be lengthened or even continued year round. This would spread fixed costs of equipment operation over more work units, reduce or eliminate layoffs of construction labor, and permit earlier use of new facilities.

MATERIALS AND TESTING PROCEDURE

Materials

The soil selected for the laboratory investigation was one of marginal frost susceptibility. It was obtained from a site near the Houghton County Airport in Michigan. The uniformity of the stored material was confirmed by grain size analysis of selected samples. Typical results of these tests are shown in Figure 1. Texturally the soil is classified as a silty sand. The unified engineering classification is SM. According to the Corps of Engineers criteria, the frost susceptibility classification is F2(b), low to medium. The specific gravity of the soil is 2.69.

The additive used in the test program was a high-test flaked calcium chloride. Solubility and freezing point depression characteristics for pure calcium chloride can be obtained from the Handbook of Chemistry and Physics (14).

Testing Procedure

When compaction tests were conducted at 68 F (20 C), the soil was compacted according to the procedure outlined in AASHO T-99 for standard compactive effort or AASHO T-180 for modified compactive effort. If an additive was used, it was added with the water. After mixing, the prepared soil was sealed in plastic bags and stored at room temperature for 20 hours prior to compaction. Compaction of the soil was carried out by using a Soiltest mechanical compactor with a 6-in. (152-mm) mold.

The testing procedure at temperatures below 32 F (0 C) involved a number of deviations from the conventional method of sample preparation and testing. To eliminate the effect of particle size, particles of constant size were prepared in the laboratory by forming individual particles in an ice cube tray. The tray contained 36 tapered cubes with an edge dimension of 0.8 in. (20 mm). Based on water content each particle was subjected to a static load by a soil loading press to achieve a precompaction density of 85 lb/ft^3 (1360 kg/m^3). The prepared samples were exposed to below-freezing temperatures for 20 hours prior to testing. Moisture loss was prevented by sealing the prepared samples in plastic bags.

Compaction in the cold room began by removing the frozen soil cubes from the trays. The cubes were then placed in the compaction mold, and they were compacted to the required energy per layer. When compaction was completed, the soil and mold were weighed. If no additive was used at the time of preparation, trimming of the frozen compacted samples was extremely difficult. In this situation the volume of the compacted sample was determined by using the sand cone method suggested by Campen (2). When 2 or 3 percent calcium chloride was used in preparing the sample, the pore water was

Figure 1. Typical gradation curve for the test soil.

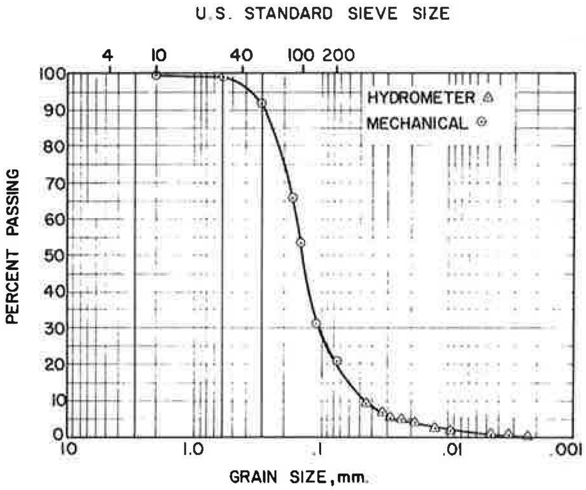


Figure 2. Moisture-density relationship for the test soil with no additive.

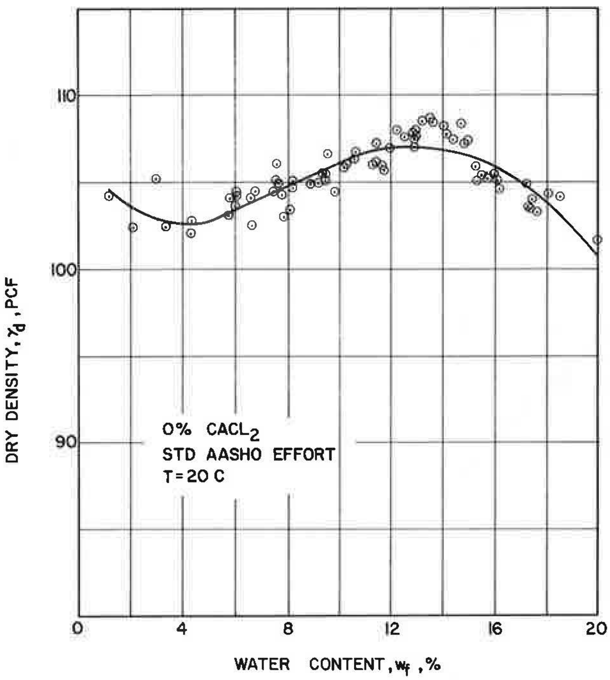


Figure 3. Moisture-density relationship for the test soil with 1 percent calcium chloride.

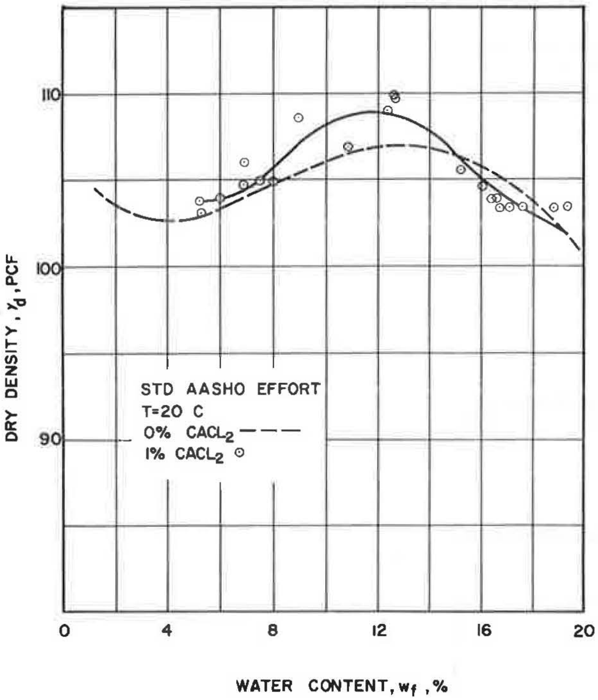
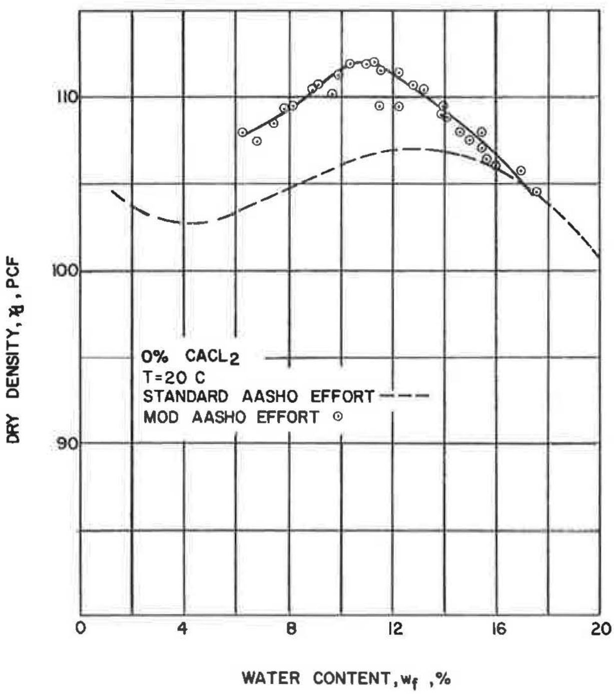


Figure 4. Moisture-density relationship for the test soil using modified AASHO compactive effort.



not completely frozen and the volume of soil was determined by conventional trimming as in the AASHO test method. Three samples from each compacted specimen were taken to determine the average moisture content.

DISCUSSION AND INTERPRETATION OF RESULTS

Compaction at 68 F (20 C)

The initial phase of the testing involved compacting the soil at room temperature to establish a frame of reference for the remaining parts of the test program. The results of standard AASHO testing are shown in Figure 2. The moisture-density curve has the shape expected for a fine silty sand. The maximum dry density is approximately 107 lb/ft^3 (1712 kg/m^3) at a moisture content of 13 percent. The increase in dry density at low water contents is typical of a soil with a large amount of fine sand (bulking) and is related to the interruption of capillary forces and the large effective stresses that go with these forces.

Seed, Mitchell, and Chan (9), in discussing the development of effective stresses in compacted soils, divided pore pressure into two parts, pore air pressure and pore water pressure. On the dry side of optimum at low water contents, the pore air pressure is low and less than the negative pore water pressure. Therefore, the total pore pressure is negative, which causes high effective stresses and resulting low densities. As the water content increases to optimum, the negative pore water pressures become increasingly less negative and the pore air pressure increases. This causes a decrease in effective stress and higher density in the compacted soil. Beyond the optimum water content, the pore air pressure is approximately constant (7), but the pore water pressures become more negative as in an undrained test on a dense, saturated soil. The large negative pore water pressure causes increased effective stress and a decrease in density typical of soils compacted on the wet side of optimum.

When an additive such as an inorganic salt is added to the soil, the basic mechanism of compaction will be the same. However, the addition of the salt may have other effects that cause the compaction characteristics of a treated soil to be different from those of an untreated soil (5, 10, 16). Johnson and Sallberg (6) concluded, "There is general agreement on the effect of calcium chloride on soils that are essentially granular. These soils have shown a consistent increase in dry unit weight due to the use of calcium chloride." In their study of the use of calcium chloride, the increases due to the additive were quite small (less than a 2 percent increase in dry density). Several test series were conducted to verify the results of work done by others and to note the effect of the additive on the soil type used in this program. Room temperature tests using standard AASHO compactive effort were conducted by adding 1.0 percent calcium chloride. The effects of the calcium chloride on the compaction characteristics are shown in Figure 3. In comparison with the untreated soil tested at the same temperature, the maximum density of the treated soil is 2 lb/ft^3 (32 kg/m^3) higher and the optimum moisture content is 2.0 percent lower.

Increasing compactive effort will result in an increase in dry density and a decrease in optimum water content for most soils. Figure 4 shows a plot of dry density versus water content for a series of tests on specimens compacted by using modified AASHO compactive effort. The resulting maximum dry unit weight is 112 lb/ft^3 (1792 kg/m^3) at a water content of 11.0 percent. Compared to the standard effort curve, the increase in dry density is modest but does follow the usual pattern of moving up and to the left.

Compaction at 19.4 F (-7 C) Without an Additive

For a soil with no additives and a temperature low enough to ensure complete freezing of all pore moisture, the shear strength of the soil is dependent not only on the frictional resistance of the soil but also on the shear resistance provided by the pore ice. It is well known that the strength of ice is highly time dependent (13), but for the short loading time involved in compaction procedures it is reasonable to assume that the resistance to compaction provided by the ice is constant. If this observation is valid, then densities obtained from compacting frozen soils will be less than those obtained from unfrozen soil by an amount proportional to the energy required to overcome the additional resistance to rearrangement provided by the ice in the pore spaces.

Several researchers (12, 17) have demonstrated that the shear strength of soil-ice materials is dependent on the amount of ice in the void spaces. By extension, it may be possible to say that the density of a compacted frozen soil should also be related to the amount of ice in the pore spaces. This implies that the density of a compacted soil-ice system will decrease as the water content increases, as has been noted by Johnson and Sallberg (6). The validity of this observation can be checked by compacting a soil frozen at various water contents, calculating the amount of ice in the sample, and plotting the results.

The results of a series of standard AASHO tests on an untreated soil frozen prior to compaction are shown in Figure 5. In this figure the temperature at which the soil was prepared and compacted was 19 F (-7 C). This temperature was low enough to ensure complete freezing of all pore fluid since the absorbed layer of water is minimal in a silty sand. The resulting curve of dry density versus water content is bilinear with the intercept at a water content of approximately 3 percent.

The behavior of the untreated soil at low temperatures can be explained by considering the amount of ice present in the void spaces at a particular water content. For example, at zero water content the particles are in contact over a very small area and no ice is present; the resistance to rearrangement is due to the frictional resistance at the soil particle contacts. The resulting dry density is nearly the same as that for a soil compacted at temperatures above 32 F (0 C). As the water content increases from zero, individual particles begin to accumulate water in the region of the intergranular contacts, and as the temperature is decreased the water changes to ice. The ice between adjacent particles increases the resistance to rearrangement by an amount proportional to the area of the ice. This causes large decreases in dry density until all particles are joined by a continuous layer of ice. This condition occurs at approximately 3 percent water content for the silty sand. Above this point, the interparticle void spaces begin to fill with ice, and the net increase in area of resistance along a potential plane of sliding is increased by only a small amount. The result is a smaller decrease in dry density as shown in the second part of the curve in Figure 5. The decrease in dry density in this region is continuous until all void spaces are filled with ice. Any further increase in the amount of ice beyond that required to fill all void spaces will result in soil particles being forced apart, which completely eliminates the frictional resistance of the soil. Compaction of the material in this state would be dependent only on the ability of the ice matrix to resist the applied energy.

Compaction at 19 F (-7 C) With an Additive

When the temperature of the soil is lowered, the pore fluid becomes more viscous and dry densities decrease slightly until the temperature of the pore fluid is lowered to the freezing point. With a further decrease in temperature, the pore fluid begins to change to ice, causing a substantial decrease in dry unit weight. However, using an appropriate amount of additive will lower the freezing point of the pore fluid, and the soil will remain unfrozen even at very low temperatures.

When calcium chloride is dissolved in water, the freezing point of the resulting solution (a function of the composition of the solution) can be determined directly from the Handbook of Chemistry and Physics. To apply freezing point depression characteristics to a compacted soil requires that the composition of the pore fluid be calculated for various levels of treatment with the additive. From the definition, the composition of a solution, A, is the weight of anhydrous compound per 100 g of solution. For a compacted soil this can be expressed as the weight of additive divided by the

Figure 5. Low-temperature dry density versus water content for soil without additive.

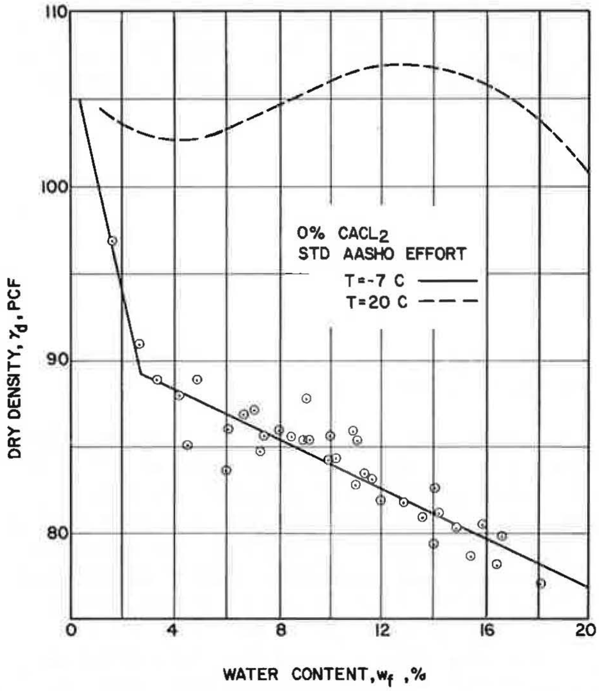
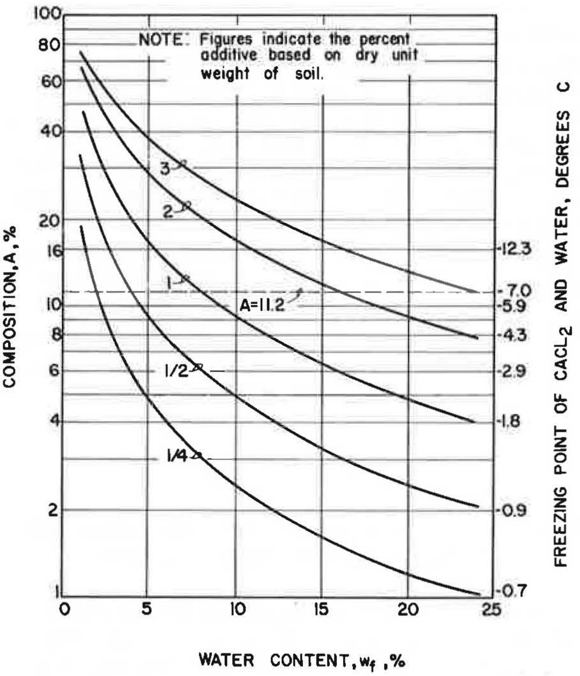


Figure 6. Composition of the soil pore fluid for various percentages of additive.



weight of pore fluid multiplied by 100. Because the amount of additive used for the compaction test series is based on the dry unit weight per cubic foot, the weight of additive is the dry unit weight of the soil multiplied by the percentage of additive divided by 100. If complete solubility of the additive is assumed, the weight of the solution is equal to the weight of water (water content times dry unit weight) per cubic foot (cubic meter) plus the weight of the additive. These observations can be substituted into the definition of A with the following result:

$$A = \frac{P_a}{w_f + P_a} \times 100 \qquad (1)$$

where

P_a = percentage of additive by weight of dry soil and
w_f = final water content of the compacted soil in percent.

This equation is independent of type of additive and can be solved for various water contents and percentages of additive. The solution to equation 1 for several different amounts of additive is shown in Figure 6. It is apparent from this graph that for a given percentage of additive the composition of the resulting solution decreases rapidly with an increase in final water content. Also shown in this figure is the composition (A = 11.2) of the solution of water and calcium chloride required to reduce the freezing point of the solution to the test temperature of 19 F (-7 C). For any combination of water content and percentage of calcium chloride that falls above this line, all pore fluid will remain unfrozen, and the compaction characteristics of the soil should remain essentially the same as an untreated soil compacted at temperatures above 32 F (0 C). For example, a soil treated with 2 percent calcium chloride that has a water content less than 16 percent will have unfrozen pore fluid.

To verify the hypothesis stated above, standard AASHO tests at 19 F (-7 C) were conducted on the silty sand after it was treated with 2 and 3 percent calcium chloride. The results of these tests are shown in Figure 7. For the tests with 2 and 3 percent calcium chloride, the maximum dry densities are essentially the same at approximately 105 lb/ft^3 (1680 kg/m^3). Also, this value is very close to the maximum dry density of the untreated soil tested at a temperature of 68 F (20 C) as was originally assumed. Additional results from modified AASHO compaction tests conducted at 19 F (-7 C) with 2.0 and 3.0 percent calcium chloride are shown in Figure 8. Compared to the standard AASHO test at the same temperature there are significantly more scatter in the dry densities and an apparent slight increase in dry density for the soil treated with 3 percent calcium chloride. The maximum dry density for the soil treated with 2 percent calcium chloride is 101 lb/ft^3 (1616 kg/m^3). This value is less than the maximum dry density obtained from the standard AASHO tests at the same temperature and is not consistent with the observation that increased compactive effort causes increased maximum dry densities. However, the maximum dry density is substantially greater than would be obtained for an untreated soil, which indicates the beneficial effect of the calcium chloride treatment.

FIELD APPLICATIONS

In the field, the process of compacting soils at below-freezing temperatures is dependent on implementation of special application, mixing, and treatment sequences in order to prevent freezing of the soil.

Depending on the moisture content of the soil, calcium chloride can be applied as either a liquid solution or dry flakes. Below optimum moisture content, a calcium chloride solution of the correct composition is the most efficient method of applying the additive. If the soil possesses sufficient moisture, application of the additive in

Figure 7. Effect of calcium chloride on the test soil compacted at 19 F (-7 C).

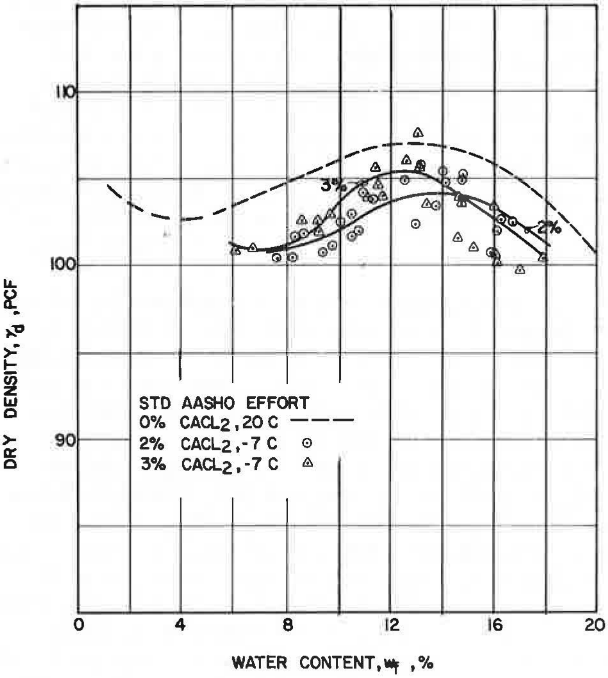
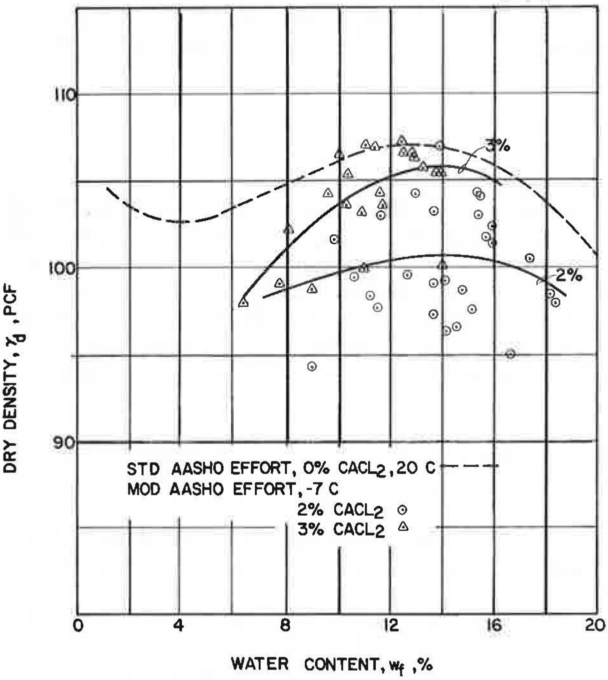


Figure 8. Effect of calcium chloride and compactive effort on the test soil compacted at 19 F (-7 C).



its dry form would be required to keep the soil moisture at optimum.

To obtain results comparable to those obtained in the laboratory requires that the additive and soil be thoroughly mixed. Common stabilization equipment, such as single- and multiple-rotor and traveling mixer units, can be effectively used. Less efficient mixing can be achieved by blading the soil and additive back and forth with a motor grader until the chemical is completely mixed and dissolved into the soil pore fluid. Agriculture equipment, such as the disk harrow, could prove useful as a mixing tool, and effective mixing of lime has been achieved (11) by spreading the lime over a surface that has been ripped by a crawler tractor and mixed with a plow. The degree of mixing that can be achieved in the field is not known, but Ingles and Metcalf (4) have concluded that a coefficient of variation of 30 percent in additive content can be expected for normal field mixing techniques. This compares with 10 percent for average laboratory work.

If good field results are to be obtained, the treatment sequence must be planned to provide calcium chloride to the soil at the proper time and in the correct amount to prevent freezing of the soil pore water. This requirement may force treatment at both the project site and the borrow area. At the borrow area it may be necessary to apply calcium chloride just prior to stopping the day's operations to prevent nighttime freezing of the borrow material. Overnight salt treatment may also be needed at the project site in the areas disturbed by the day's construction activities. Normally, calcium chloride treatment is required during daily earthwork operations only if temperatures are low enough to cause freezing prior to final placement and compaction of the soil. Surface treatments during the day can be minimized by keeping the work areas as small as possible and working in the vertical direction of the embankment or borrow area.

Various problems other than those listed above are associated with field compaction of soils at below-freezing temperatures. Large temperature fluctuations can make it difficult for the field engineer to make a decision on whether treatment is required and how much. Precipitation in the form of snow presents a maneuverability problem for the mixing equipment and reduces the effectiveness of treating the soil with calcium chloride. However, these problems do not diminish the fact that a soil properly treated with calcium chloride can be prevented from freezing and can be successfully compacted. Careful cost analysis is required for each project to determine whether the costs of calcium chloride, including mixing with the soil, would be offset by improved equipment utilization, reduction in seasonal layoffs of labor, and earlier use of the new facility.

CONCLUSIONS

The experimental program conducted as part of this study required a large number of compaction tests on a single soil at various temperatures and compactive efforts. Of primary interest were the compaction tests at 19 F (-7 C), but a substantial number of tests were conducted at 68 F (20 C) to establish a frame of reference for the tests at lower temperatures. An additive was used to investigate possible methods of improving compaction of soils at low temperatures. Based on the results of the experimental program the following conclusions concerning the compaction of soil at low temperature were made.

1. The dry unit weight of compacted frozen soil is less than the dry unit weight of a soil compacted with the same effort but at a temperature above the freezing point of the pore fluid.
2. The dry unit weight of compacted frozen soils is inversely proportional to the amount of ice in the pore space. For the soil tested, the relationship between frozen dry weight and water content is bilinear.
3. Additives can be effectively used to alter the compaction characteristics of a soil prepared and compacted at temperatures below 32 F (0 C).
4. If enough additive to depress the freezing point of the pore fluid below the test temperature is used, the compaction characteristics of a soil tested at a temperature

below 32 F (0 C) will be the same as those of a soil without an additive compacted at temperatures above 32 F (0 C). For soils compacted in this state the optimum water content of the treated soil is close to the optimum obtained for an untreated soil tested at temperatures above 32 F.

5. The amount of additive required to prevent freezing of the pore fluid can be obtained from the freezing point depression characteristics of the additive. For calcium chloride the values are shown in Figure 6.

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VARIATION IN LABORATORY AND FIELD STRENGTHS OF SOIL CEMENT MIXTURES

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This report evaluates the variability in compressive strengths of stabilized in-place soil cement mixtures from the standpoint of design and actual field conditions. The findings are based on 15 projects with soils ranging from high silt to high sand content and 8 to 14 percent cement by volume. The data indicate considerable variation in the laboratory and field-molded specimens. In general, under the present construction techniques of cement application and density and moisture control, the product is within 75 percent of the 28-day design strength (225 psi or 1550 kPa). The data also indicate a need for pug mill mixing of soil and cement to reduce cement content variation.

•THIS report, which is an abridgment of a comprehensive study (1), evaluates the variability in compressive strengths of stabilized in-place soil cement mixtures from the standpoint of design and actual field conditions. The evaluation is based on 15 projects with soils ranging from high silt to high sand content and plasticity indexes of up to 15. The cement content varied from 8 to 14 percent by volume.

PROCEDURE

For strength evaluation, specimens were prepared and cured both in the laboratory and in the field. The strengths of these specimens were compared to the 7- and 28-day strengths of roadway cores. The variability of the laboratory design was studied in four phases. Each phase was designed to provide data on the variability among and within laboratories.

TEST RESULTS

Figure 1 shows the mean percentage of all tests that achieved specified compressive strengths in each mode of sampling and curing.

The means for the first bar chart (laboratory-molded and laboratory-cured, 7 day) include compressive strength results of materials in which the cement quantity recommendations were originally based on the wet-dry brush test, as well as those actually based on 300 psi (2070 kPa). The projects in which compressive strength was used for cement recommendations show substantial verification of the materials laboratory design, with only one of these projects having soil types in which less than 300 psi was obtained at the recommended cement percentage.

Variability of Cement Content of Bases

Cement contents of field-molded specimens and cores were determined on all 15 projects.

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Figure 1. Mean percentages of specimen compressive strengths.

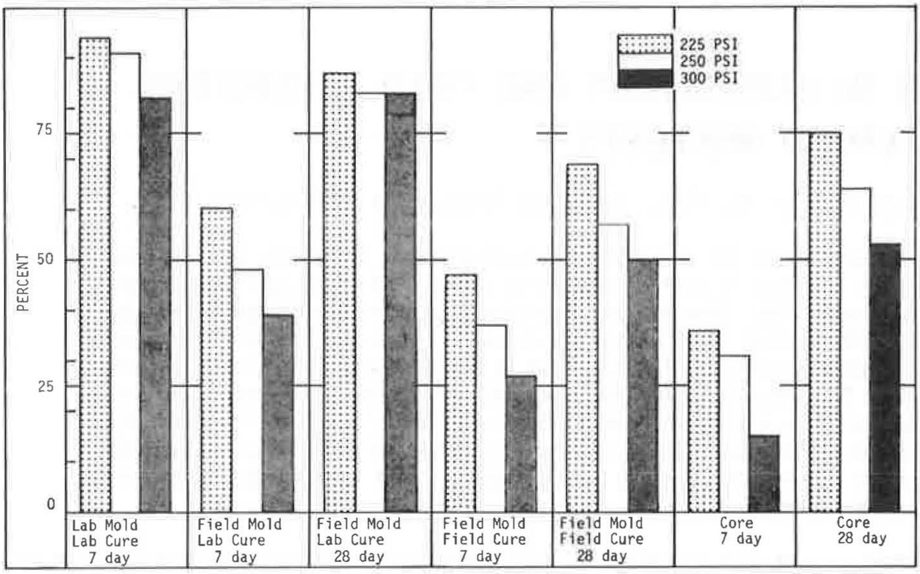
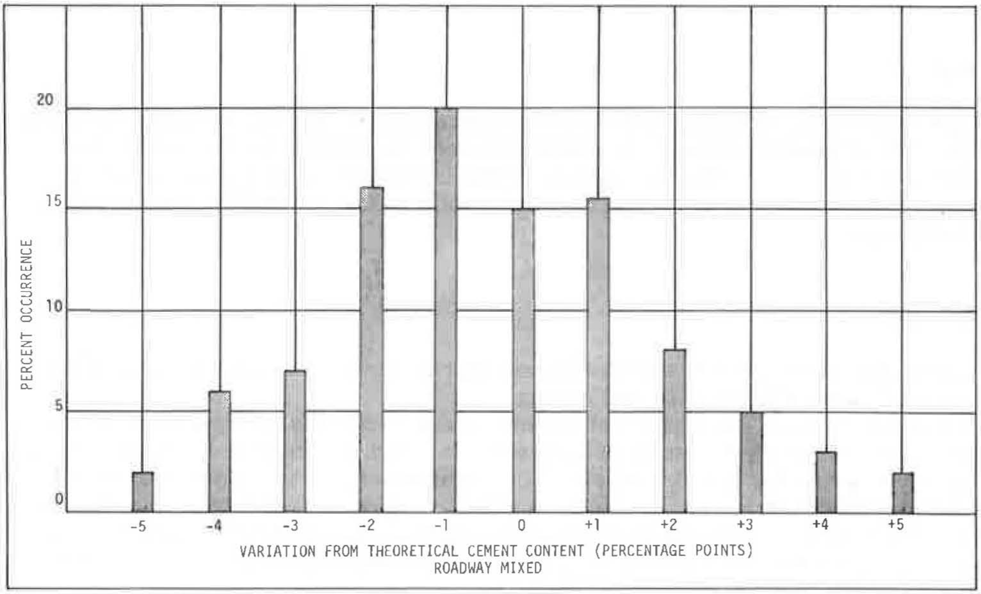


Figure 2. Frequency of occurrence of actual and theoretical cement contents.



An attempt was made to correlate the cement content of field specimens with strength; however, a definite trend could not be established because of the variation in specimen density and curing. These test results, however, did show the wide variation of cement within the soil cement bases as a result of in-place mixing. This is shown in Figure 2, which is a composite of all test runs (311 observations) and shows the percentage of observations in which the roadway-mixed cement content varied from the theoretical cement content.

Variability in Laboratory Design

In the process of obtaining the laboratory data, it was discovered that the laboratory design procedure for soil cement, based on compressive strength, exhibited a greater amount of variability than was previously acknowledged. At first, procedural errors were blamed, but repeated tests under strictly controlled circumstances confirmed the degree of variability. It was found that a difference of 100 psi (6890 kPa) between identical specimens could occur.

To evaluate the variability of the test procedure, interlaboratory tests were conducted. Such test procedures are used to ascertain whether a product meets the specifications set down for the products, or they may be performed for design purposes as was the case here. Regardless of the purpose, the information desired is whether the test procedure as set forth is capable of yielding acceptable agreement among results from different laboratories.

Table 1 gives the statistical parameters for the first phase of the cooperative test. Laboratory designations used in Table 1 are as follows:

Designation	Laboratory
1	Research
2	Materials
3	District 07

The variation for each series of soil cement data is expressed by the standard deviation. In the comparison of variability among laboratory-soil cement series data, the relative measure of the dispersion is given in the table as the coefficient of variation, which is the ratio of standard deviation to the mean of a given series. This measure is particularly useful when widely differing means are encountered.

The magnitude of the coefficient of variation for the district laboratories was considerably higher than that indicated by the research or materials laboratories. This was because they were unfamiliar with the test procedure. Furthermore, the magnitude of this variation was considerably higher than would be expected because of chance alone. Therefore, an effort was made to isolate the causes of variation before the second round of cooperative testing commenced.

It was found that the temperature of the three components—soil, cement, and water—varied widely within and among the three laboratories. The soil and the cement were stored, in some instances, in areas where temperatures were not controlled. That is, the temperature of the storage area fluctuated with the season: high in summer and low in winter. This could result in the use of hot cement and soil for some specimens molded in the summer.

Tap water was used in the molding of all specimens. This in itself did not seem to cause any problems; however, the temperature of one laboratory's tap water was close to 100 F (38 C) because its pipes were adjacent to the building's steam lines.

A check of specimens immediately after molding revealed many dry particles. The existing procedure required the full incorporation of water and cement immediately

Table 1. Statistical evaluation of laboratory data before standardization.

Soil	Cement Content (percent)	Mean 1	Mean 2	Mean 3	Standard Deviation 1	Standard Deviation 2	Standard Deviation 3	Coefficient of Variation 1	Coefficient of Variation 2	Coefficient of Variation 3
A	6	289.33	220.00	254.13	14.36	9.26	21.72	0.05	0.04	0.08
	8	414.00	344.25	344.88	26.41	34.31	22.63	0.06	0.10	0.07
	10	548.67	439.13	492.88	20.13	19.06	25.09	0.04	0.04	0.05
	12	678.33	698.25	647.00	59.80	36.62	65.42	0.09	0.05	0.10
	14	789.25	830.78	871.88	60.29	60.33	136.43	0.08	0.07	0.16
	16									
B	6	211.63	–	–	10.41	–	–	0.05	–	–
	8	278.38	207.88	208.00	14.60	14.20	29.39	0.05	0.07	0.14
	10	315.88	219.63	289.25	8.87	5.90	25.19	0.03	0.03	0.09
	12	387.25	303.63	304.00	17.19	17.53	24.91	0.04	0.06	0.08
	14	442.88	335.88	357.25	30.76	28.82	28.85	0.07	0.09	0.08
	16	465.63	411.88	427.38	11.06	16.47	46.49	0.02	0.04	0.11
C	6	277.75	206.00	231.38	12.12	15.55	26.75	0.04	0.08	0.12
	8	344.38	279.13	283.75	23.65	14.37	30.65	0.07	0.05	0.11
	10	398.25	309.25	349.63	15.94	13.11	53.03	0.04	0.04	0.15
	12	445.00	356.50	479.55	24.04	23.40	55.75	0.05	0.07	0.12
	14	535.63	399.65	469.50	49.51	37.40	36.15	0.09	0.09	0.08

Note: 1 psi = 6.9 kPa.

Table 2. Statistical evaluation of laboratory data after standardization.

Soil	Cement Content (percent)	Mean 1	Mean 2	Mean 3	Standard Deviation 1	Standard Deviation 2	Standard Deviation 3	Coefficient of Variation 1	Coefficient of Variation 2	Coefficient of Variation 3
Sandy loam	6	353.00	237.25	219.00	17.98	8.61	18.40	0.05	0.04	0.08
	8	476.50	302.50	281.75	45.54	15.69	2.36	0.10	0.05	0.01
	10	538.00	345.75	322.00	47.22	5.12	20.49	0.09	0.01	0.06
	12	654.25	445.75	440.25	22.31	29.68	27.40	0.03	0.07	0.09
	14	820.25	506.75	556.75	48.04	8.83	52.71	0.06	0.02	0.09
	16	899.00	557.00	540.25	24.22	30.60	25.32	0.03	0.05	0.05
Clay loam	6	254.25	149.75	181.25	15.73	6.65	5.19	0.06	0.04	0.03
	8	333.00	192.25	220.50	8.60	13.40	16.38	0.03	0.07	0.07
	10	359.50	220.00	227.00	16.10	8.83	12.06	0.04	0.04	0.05
	12	391.00	234.50	224.75	19.04	17.06	16.03	0.06	0.05	0.07
	14	440.50	276.50	273.75	19.33	9.25	17.04	0.04	0.03	0.06
	16	510.00	275.50	322.75	4.81	21.13	27.29	0.01	0.08	0.08
Silty loam	6	189.50	136.75	140.75	5.67	5.91	16.50	0.03	0.04	0.12
	8	214.00	167.00	159.50	18.78	12.83	14.11	0.09	0.08	0.09
	10	271.50	199.50	195.50	15.46	12.37	37.12	0.06	0.06	0.19
	12	329.00	218.00	216.50	13.29	11.52	6.35	0.04	0.05	0.03
	14	403.00	265.00	266.00	12.38	12.73	27.64	0.03	0.05	0.10
	16	429.75	310.00	292.25	13.33	16.79	15.31	0.03	0.05	0.05

Note: 1 psi = 6.9 kPa.

prior to mixing. The soil particles did not adequately absorb the water immediately, causing density variations. Later, during the curing process, these soil particles possibly competed with the cement for the available water.

Another possible cause of variation was the cement itself. The cement used by the three laboratories came from different sources. Seven-day compressive strength (AASHO T-106) varied from 2,100 to 4,500 psi (14 480 to 31 030 kPa).

To alleviate these possible causes of variation the following steps were taken:

1. Each component in the fabrication of soil cement specimens was brought to the same temperature, 75 ± 5 F (24 ± 3 C), before the specimens were molded;
2. Water was added to the raw soils, and the mixture was allowed to slake overnight before addition of cement;
3. Cement from the same manufactured batch was used; and
4. The time involved in fabrication of specimens was held uniform.

The densities and moisture contents of the specimens were closely controlled among the three laboratories by using the same density and optimum moisture for specimen design for each material tested.

On the basis of standardization, a second set of soil samples was distributed to the same laboratories. The soils belonged to the same classification group. The same experimental design as the first one was used in this phase except that there were four replications instead of the previous eight. The improvement in the variability is clearly evident from data given in Table 2. With the exception of two series of data, the relative dispersion was 0.10 or 10 percent or less. Overall there was a decrease in the variability of the test procedure as a result of standardization.

The effect of soil samples stored for some period of time and then mixed for cement content determination was another aspect studied. To test whether there is a difference due to time in the strength property of specimens mixed and compacted at different times by the same laboratory, the statistical t-test for unpaired data was run. The mean for each soil group data obtained at time A was compared to the mean of the same soil data obtained at time B. On the basis of the calculated t-values, none of the differences in the means was significant at the 0.05 level.

CONCLUSIONS

The primary conclusions on the basis of this study are as follows:

1. The inconsistency in laboratory design can be minimized if factors such as molding, temperatures, slaking, and fabrication time are closely adhered to.
2. Under the present construction techniques of cement application and density and moisture control, a fair product is produced with 75 percent of the stations having achieved 75 percent (225 psi or 1550 kPa) of the design strength at 28 days. The compressive strength of cores taken on eight projects after 3 months or more was well over 300 psi (2070 kPa).
3. For those projects in which the laboratory design criteria were based on compressive strength, the raw soils sampled and tested in the laboratory showed substantial verification of the materials laboratory design. Only one project had soil types in which the 7-day strength was less than 300 psi (2070 kPa) at the recommended cement percentages.
4. In-place mixing of cement with soil appears to be somewhat undesirable. Results of 311 observations show a variation of ±5 percent from the theoretical cement content in the soil cement bases studied.

ACKNOWLEDGMENT

This report reflects the views of the authors and does not necessarily reflect the official views or policies of the Louisiana Department of Highways or the Federal Highway Administration.

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A METHOD FOR QUANTITATIVE DETERMINATION OF SOIL MINERALS BY X-RAY DIFFRACTION

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ABRIDGMENT

•THIS paper describes the theory and initial test results for a method of quantitatively determining the amount of various mineral components in a soil sample by comparing X-ray diffraction peak intensities of the mineral in the sample with the intensities of the same peak in specimens that are a mixture of the original soil sample and known additional amounts of the mineral in question. That is, the component being determined is used as an internal standard.

Norrish and Taylor (2) expanded on the ideas of Klug and Alexander (1) and Von Engelhardt (4) to demonstrate that the measured intensity of an X-ray diffraction peak of a crystalline component in a sample is related to the weight fraction of that component in the sample as follows:

$$I_o = \frac{Kx}{\rho A_s} \tag{1}$$

where

I_o = measured intensity of the diffraction peak of a crystalline component in a soil sample,
ρ = true density of the component used to make the diffraction pattern,
x = weight fraction of the component being estimated,
K = constant for any particular peak of a particular component, and
A_s = mass absorption coefficient of the sample.

Norrish and Taylor pointed out that the use of this equation with internal standards is difficult because the mass absorption coefficients of the standard mixtures vary as the amount of known mineral content changes in the specimen. The derivation here takes this variable into account but assumes that the ratio of K/ρ will remain constant as long as the crystalline structure of the standard component added to the soil sample is essentially the same as that of the component occurring in the soil sample.

Equation 1 can be written as

$$I_o = \frac{Kx}{\rho(xA_m + x_2A_2 + x_3A_3 + \ldots)}$$

where

A_m = mass absorption coefficient of the component being measured,

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A_2, A_3 = mass absorption coefficients of the other minerals in the soil, and
x_2, x_3 = weight fractions of those other minerals.

The true mass absorption coefficient of the sample consisting of components exclusive of the one being measured is A_1. These minerals exclusive of the one being estimated will be referred to as the matrix minerals. The true mass absorption coefficient of the matrix minerals is given by

$$A_1 = \frac{x_2}{1 - x} A_2 + \frac{x_3}{1 - x} A_3 + \ldots$$

Therefore,

$$A_1 = \frac{1}{1 - x} (x_2A_2 + x_3A_3 + \ldots)$$

The term $x_2A_2 + x_3A_3 + \ldots$ can be called the apparent mass absorption coefficient of the matrix minerals, $\bar{A}_1$. Therefore,

$$I_o = \frac{Kx}{\rho(xA_a + \bar{A}_1)} \tag{2}$$

If a known quantity of the component being estimated is added to the sample, this mixture can be referred to as a specimen. Let c be the known added weight fraction of the component in the specimen and X be the composite or total weight fraction of the component in the specimen. The weight fraction of the sample in the specimen is 1 - c, and the unknown weight fraction of the component in the specimen is (1 - c)x. Therefore, the total weight fraction of the component in the specimen is

$$X = (1 - c)x + c$$

The peak intensity produced by the total amount of the component in the specimen is

$$I_c = \frac{KX}{\rho A_c} = \frac{K[(1 - c)x + c]}{\rho A_c} \tag{3}$$

where

I_c = peak intensity, and
A_c = complete mass absorption coefficient of the specimen.

The mass absorption coefficient in terms of the total and added weight fractions in the specimen is

$$A_c = XA_a + (1 - c)x_2A_2 + (1 - c)x_3A_3 + \ldots$$

which can also be written in terms of the unknown weight fraction of the component in the sample as

$$A_o = [(1 - c)x + c]\,A_s + (1 - c)\,(x_2A_2 + x_3A_3 + \ldots)$$

The value of the apparent mass absorption coefficient of the matrix minerals can be substituted into the above equation to give

$$A_o = [(1 - c)x + c]\,A_s + (1 - c)\,\bar{A}_1 \qquad (4)$$

Substitution of equation 4 into equation 3 gives

$$I_o = \frac{K[(1 - c)\,x + c]}{\rho\{[(1 - c)x + c]\,A_s + (1 - c)\,\bar{A}_1\}}$$

The ratio of the peak intensities produced by the sample and the specimen is

$$\frac{I_s}{I_o} = \frac{[(1 - c)x + c](xA_s + \bar{A}_1)}{\{[(1 - c)x + c]\,A_s + (1 - c)\,\bar{A}_1\}x}$$

which can be written as

$$\frac{I_s}{I_o} = \frac{\dfrac{A_s}{\bar{A}_1} + \dfrac{1}{x}}{\dfrac{A_s}{\bar{A}_1} + \dfrac{1 - c}{(1 - c)x + c}} \qquad (5)$$

If

$$\frac{A_s}{\bar{A}_1} = A$$

and this term is substituted into equation 5, then by adding and subtracting 1 the equation becomes

$$\frac{I_s}{I_o} = \frac{c/x}{Ax - Acx + Ac + 1 - c} + 1 \qquad (6)$$

Now let $\frac{I_s}{I_o} - 1 = I$ and substitute it into equation 6 to give

$$\frac{c}{I} = x^2A - Acx^2 + Acx + x - cx$$

Grouping terms gives

$$\frac{c}{I} = c[Ax - x\,(Ax + 1)] + x\,(Ax + 1) \tag{7}$$

Let $m = [Ax - x(Ax + 1)]$ and $a = x(Ax + 1)$. Equation 7 becomes

$$\frac{c}{I} = mc + a \tag{8}$$

such that, if the concentration of component added to the sample divided by the intensity ratio minus one is plotted versus the concentration of the component added to the sample for various concentrations, then a straight line with a slope of m and an intercept of a should result. From these experimentally determined values it should be possible to compute x, the unknown amount of the component in the soil sample. The values of a and m give two equations with two unknowns, x and A. Thus, it is also possible to compute experimentally the ratio of the mass absorption coefficient of the mineral A_s as well as the mass absorption coefficient of the matrix minerals.

EXPERIMENTAL PROCEDURE AND OBSERVATION

The reliability of this method was evaluated by preparing an artificial soil containing 5 percent hematite ground from a naturally occurring hematite and 95 percent kaolinite. This soil sample was then mixed with additional known amounts (5, 8, 12, 15, and 20 percent) of a chemically pure hematite. The intensities of the 100 peaks for the sample and each specimen were measured. The intensity ratio for each specimen was computed, and a plot of c/I versus c was prepared. The radiation used was molybdenum Kα, and the 100 reflection gives a peak at the 2θ angle of 15.15 deg. As shown in Figure 1, there is an overlap of the 15.15-deg peak and the next peak corresponding to both hematite and kaolinite at about 16.4 deg. Therefore, it is necessary to sketch the lower portion of both peaks to the base line so that the higher intensity caused by the overlap is equal to the sum of the areas of the two tails. The base line is determined by extending a straight line from one background level at a lower 2θ angle where there are no peaks to another low background at a higher 2θ angle. The area of the peak so defined was then measured with a planimeter.

Four intensity measurements were made on each specimen and the soil sample; then the intensity ratios were calculated by using the average intensities. The values of c/I were then calculated and plotted versus c. The graph is shown in Figure 2 with the regression line and 95 percent confidence limits. The hematite content determined from a and m is 5.15 percent, which compares favorably with the 5.00 percent of the artificial soil sample. The calculus method (3) of error analysis revealed that at the 5 percent significance level the range in calculated hematite contents is from 4.6 to 5.7 percent. Theoretical mass absorption characteristics for hematite and kaolinite were determined as 27.26 and 3.4 respectively. The theoretical value of the mixture of 5 percent hematite and 95 percent kaolinite is therefore 8.4. The mass absorption coefficient that was determined from Figure 2 and the parameters a and m is 6.11.

CONCLUSIONS

Based on the good agreement between the amount of hematite in the artificial soil sample and the amount of hematite estimated by the method described here, it is concluded that this method is a reliable method for determining various mineral com-

Figure 1. Example of definition of X-ray peak for intensity measurements.

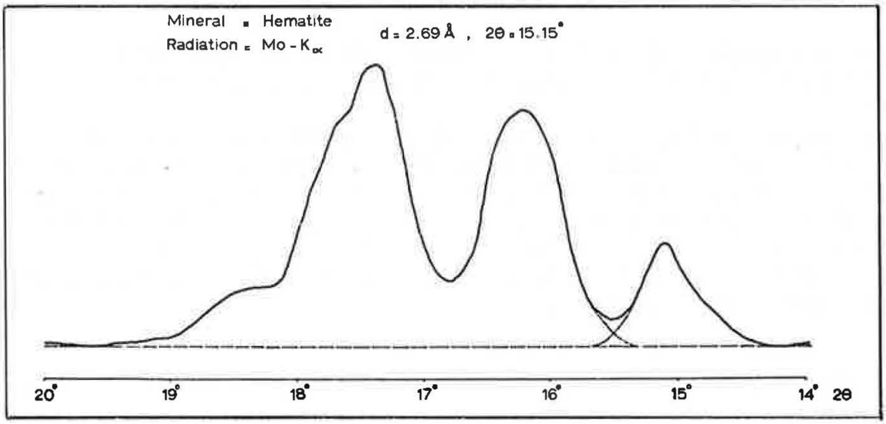
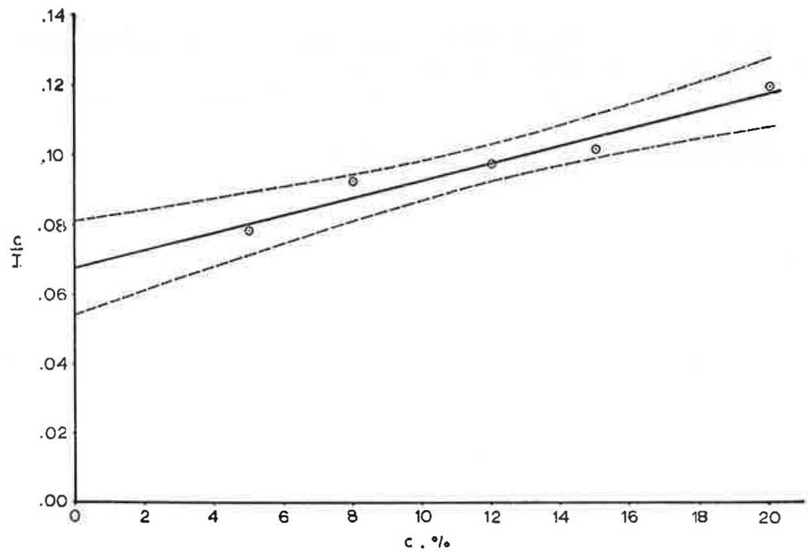


Figure 2. Concentration-intensity ratio versus concentration of hematite based on average values of peak intensity.



ponents in natural soils. The fair agreement between theoretical and experimentally determined mass absorption coefficients is taken as further evidence of the reliability of this method.

One source of the variability in the four individual values is interpreted as arising from the judgment that goes into the definition of each peak due to the overlap of sequential peaks.

Another source of scatter in the values may be the variability in the time rate of X-ray photon densities due to voltage alternation. Thus, on any given determination a different population of grains in the powder will be irradiated; this will result in varying intensities for each individual determination. Rotating the sample should minimize this effect. It is recommended that in subsequent evaluations of this technique the samples be rotated in the plane of the sample holder so as to give a better statistical sample of the crystallites being irradiated.

ACKNOWLEDGMENTS

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